Canada. Dominion Water and Power Bureau
Report on tidal power, Petitcodiac and Memramcook estuaries.
Pt.1:- Text.

DEPARTMENT OF MINES AND RESOURCES, CANADA
SURVEYS AND ENGINEERING BRANCH—DOMINION WATER AND POWER BUREAU

Report on

# TIDAL POWER

## PETITCODIAC AND MEMRAMCOOK ESTUARIES

PROVINCE OF NEW BRUNSWICK

1945

PART 1 - TEXT

OTTAWA
EDMOND CLOUTIER
PRINTER TO THE KING'S MOST EXCELLENT MAJESTY
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PART 1 - TEXT

H. G. ACRES & COMPANY
Consulting Engineers
Niagara Falls, Canada



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# FOREWORD

The possibility of harnessing the ocean tides for power at Hopewell Cape in New Brunswick, where the estuaries of the Petitcodiac and Memramcook Rivers join at the head of Shepody Bay, has been in the minds of the people of that region for many years. At this locality the tides have ranges varying from 21 to 52 feet and the two estuaries offer the possibility of a two-basin scheme which could produce continuous power.

Impressed with the possibilities of this scheme as a source of low-cost electric energy for the Maritime Provinces, representations were made on March 22, 1944, by a delegation from that region to the Railways, Telegraphs and Harbours Committee of the Senate in support of an investigation by competent engineers to determine the practicability of the development as a post-war project.

A resolution favouring such an investigation was adopted by the Senate and subsequently the Governments of the Dominion and of the Province of New Brunswick agreed to share the cost. Necessary arrangements for the investigation were made by the Department of Mines and Resources and by Order in Council P.C. 5346 of July 13, 1944, the Minister of that Department was authorized to enter into an agreement with H. G. Acres and Company, Consulting Engineers, Niagara Falls, Ontario, to undertake surveys and investigations and to submit a complete and conclusive report on the tidal power possibilities at the confluence of the Petitcodiac and Memramcook Rivers.

Field investigations by H. G. Acres and Company were commenced early in August, 1944, and were completed in late December. At the same time office studies were inaugurated of the many complex and unprecedented problems presented by a tidal power development and these were carried on continuously until October, 1945, when the report was completed. Early in November copies were made available to the two governments for consideration. The report was tabled in the House of Commons, November 19, 1945, by the Honourable J. A. Glen, Minister of Mines and Resources, and in the Senate, December 7, 1945, by the Honourable W. McL. Robertson.



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DEPARTMENT OF MINES AND RESOURCES, CANADA

SURVEYS AND ENGINEERING BRANCH

November 8, 1945.

DEAR SIR,—I am submitting herewith for consideration the report on the development of tidal power at the Petitcodiac and Memramcook River Estuaries in the Province of New Brunswick, and which has been prepared by H. G. Acres and Company, Consulting Engineers, acting on instructions transmitted through the Dominion Water and Power Bureau of this Branch.

Faithfully yours,

J. M. WARDLE,
*Director.*

Dr. CHARLES CAMSELL, C.M.G.,
Deputy Minister,
Department of Mines and Resources,
Ottawa, Ontario.

H. G. ACRES & COMPANY

*Consulting Engineers*

NIAGARA FALLS, CANADA

November 3, 1945.

VICTOR MEEK, Esq., Controller,
Dominion Water and Power Bureau,
Surveys and Engineering Branch,
Department of Mines and Resources,
Ottawa, Canada.

DEAR SIR,—We have the honour to transmit to you our report on the development of power from the tides at the confluence of the Petitcodiac and Memramcook Rivers in the Province of New Brunswick, Canada.

The engineering investigations described in this report were commenced and carried out nearly to completion under the direction of the late Dr. Henry G. Acres, founder of the firm of H. G. Acres and Company, and were sufficiently advanced at the time of his unfortunate demise that the conclusions herein given were abundantly clear to him. A sketch outline of them had been commenced by Dr. Acres just prior to his final sickness.

These conclusions, as related to the particular site which we investigated at your request, have been summarized in Section 14 of our report. We respectfully direct your attention to them.

Should there be any part of the subject matter of the report concerning which you desire amplification or additional information, we should be very happy to serve you at any time.

We wish to express our deep appreciation of the confidence which you have placed in us and our great pleasure in having had the opportunity of carrying out an investigation of such importance and interest.

Yours respectfully,

H. G. ACRES AND COMPANY,
A. W. F. MCQUEEN,
*Hydraulic Engineer.*



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## TABLE



## APPENDICES



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NOTE.—All elevations given in this report are referred to the Geodetic Datum of the Department of Mines and Resources, Canada, the zero of which is based on the determination of mean sea level at Halifax, N.S.

# REPORT ON TIDAL POWER
# PETITCODIAC AND MEMRAMCOOK ESTUARIES
# PROVINCE OF NEW BRUNSWICK
# 1945

## 1. UNDERLYING PRINCIPLE

In theory any tide, anywhere, can produce power in some quantity from the energy otherwise dissipated during its ebb stage, but such development is only practically feasible when a topographical condition obtains which (*a*) causes the incoming tide to rise to an unusual or abnormal height through being forced to its culminating level in a long gradually contracting channel, and (*b*) permits a stable barrier to be placed in the path of the tide, provided with openings to allow the tide flow to pass through to its culminating level, and thereafter capable of being closed to hold this level as the tide recedes on the seaward side of the barrier. Under such a condition it follows that as the tide naturally recedes seaward, a gradually increasing head differential is created between the falling levels and the fixed level held above and within the barrier. When this head differential has reached some pre-determined practical value, the potential energy thereby created, through the agency of the tidal flow impounded by the barrier, can be converted into controllable mechanical power. The limitations associated with the production of this power are, first, that there will necessarily be a constantly varying rate of power potential within the recurring 24-hour cycle of ebb and flow, and second, a similarly recurring cessation of all power production during a period which begins when the rising tide, coupled with the falling level in the impounding basin, has obliterated the operable head, and ending when the tide has receded to a level at which the predetermined minimum operating head can function, and the impounding basin is again full to high slack level. In principle, therefore, a tidal development can only produce power intermittently within the daily tide cycle, and auxiliary power must be obtained from some other source to ensure the continuity of service which commercial usage requires.

The world's outstanding exemplification of a topographical conformation which lends itself to a physically feasible tidal power development is the Bay of Fundy. Here, at Hopewell Cape, at the head of Shepody Bay, one of its main subsidiaries, the estuary of the Petitcodiac River, at its point of entrance into Shepody Bay, is subject to a cycle of tidal range, under normally quiescent weather conditions, varying from a minimum of 21 feet to a maximum of 52 feet.

A further topographical feature is associated with this site, which, in conjunction with the abnormally high tide, makes it unique, so far as is known, as an opportunity for tidal power development. This further condition arises from the fact that the site in question actually includes two tidal estuaries lying side by side within an out-to-out distance of 2 miles. Between the estuary of the Petitcodiac River and the estuary of the Memramcook River lies a high narrow ridge of sandstone ledge which narrows down to a sharp, steep, rocky promontory known as "Fort Folly Point", separating the mouths of the two estuaries. (See map—Plate No. 1). Later in the discussion it will be explained how these two adjacent tidal basins can be made to produce continuous power from the rise and fall of the tides prevailing at that point.

In the past many tidal power schemes have been the subject of discussion and speculation, but in only two instances, so far as is known, have any organized

investigation and study been attempted. One of these is in the estuary of the Severn River on the west coast of England, and the other is the more important and better-known "Passamaquoddy Scheme" on the coast of Maine. The minimum head differential in the case of the Severn Scheme was estimated to be 5 feet and in the case of Passamaquoddy 6 feet.

The two-basin scheme for the production of continuous power, which forms the subject of this report, is therefore without any known precedent either as to detailed engineering design and layout, or as to subsequent construction and operation.

## 2. BASIC ELEMENTS OF PROPOSED DEVELOPMENT

In order that a mental picture of the general scheme may be formed at the outset to aid in the understanding of what follows hereafter, a brief description of its fundamental elements is desirable at this point. This description is to be read with reference to Plate No. 2.

By way of explanation, the distinction between the terms "operable head" and "operating head", as used in this discussion, had best be explained.

"Operable head" is a factor which is independent of the site and its topography, and in this discussion simply means the minimum head under which any turbine, having a type and capacity in rational harmony with the project for which it is to be used, can operate with a reasonable degree of efficiency.

"Operating head" is a factor directly related to the topographical and hydraulic characteristics of the site, and is derived from a study of these characteristics.

In the first place, this plan shows the topographical relationship between Shepody Bay, the Petitcodiac estuary, and the estuary of the Memramcook, with the previously-mentioned ridge and promontory separating the two estuaries.

At the extreme left of this plan is indicated a lock with approach channels, next a rock-fill dam, next in order a concrete bulkhead containing control gates, and beyond this bulkhead, and in line with it, a second rock-fill dam, and in line with this latter structure a second concrete bulkhead, also containing control gates, and abutting the shore of the Memramcook estuary. As will be seen, this series of structures completely spans the mouths of the two estuaries.

Finally, projecting upstream roughly parallel with the inner shore of the Memramcook estuary will be seen a third bulkhead structure which represents the power-house section of the development.

As shown, these structures create or define three separate basin areas; namely, the headwater basin in the Petitcodiac estuary, the inner tailwater basin in the Memramcook estuary, and the outer tailwater basin at open sea level in Shepody Bay. The arrows indicate the direction of flow when the powerhouse and control gates are either functioning simultaneously or in sequence.

In the case of the conventional intermittent type of tidal power development the power-house structure would be an integral part of the rock-fill dams across the Petitcodiac estuary, and the spent water from the turbines would discharge directly to open sea level in the outer tailwater basin. The fact that the tailwater is not so discharged constitutes the special and unique feature of the Petitcodiac development scheme, as will now be explained.

Assuming the procedure of placing the plant in operation to begin during the slack period of low tide, the inlet gates in the Petitcodiac dam would be opened, the power-house gates would be closed, and the outlet gates in the Memramcook dam would be closed. As the incoming tide rises against the barrier structures across the two estuaries, the tide level will in due course submerge the open inlet gates. The filling of the headwater basin commences at this point and continues until the tide reaches its culminating level for that day.

When this culminating level has been reached, the inlet gates are closed, thereby impounding the tidal flow within the estuary area. Meanwhile the inner tail-water basin holds at or near low tide level, thus creating for any one tidal cycle the maximum head differential available for the production of power.

At, or somewhat previous to the culminating point of tidal inflow the turbine gates in the power-house are opened, and the turbines begin to draw impounded water from the headwater basin and to discharge their tailwater into the inner tailwater basin, the outlet gates of which remain closed for the time being. The discharge of tailwater into this inner basin during the higher stages of ebb and flow is the special feature of operation, inherent in the topography of the site, which makes the production of continuous power physically feasible.

It will, of course, be understood that while the turbines are discharging into the inner basin with the outlet gates closed, the water in this basin is gradually rising. Meanwhile, however, the tide has begun to recede in the outer tailwater basin and in due course a point is reached when the water levels in the inner and outer tailwater basins coincide. When this equalization point is reached the outlet gates are opened and the turbines thereafter discharge through the outlet gates into the free levels of the outer tailwater basin. This period of free discharge continues for the remaining duration of the ebb, and until the succeeding tidal inflow reaches a level which would cause the water to flow into, instead of out of the inner tailwater basin. When this point is reached the outlet gates are again closed and the tailwater discharge begins to accumulate in storage in the inner basin, while coincidently the rising tide submerges the inlet gates, which are opened when the rising inflow level equalizes with the falling drawdown level in the headwater basin. The filling of the headwater basin then continues to the culminating level of the incoming tide, when the inlet gates are again closed. In the meantime the turbines continue to discharge into storage in the inner tailwater basin until the ebb has carried the level of the outer tailwater basin down to a level coincident with the raised storage level of the inner basin, at which point the outlet gates are again opened for free discharge into the outer basin, and the 24-hour cycle of operation necessary for the production of continuous power has been completed.

For the immediate purpose of the above discussion, it will be sufficiently accurate to state that the amount of primary or continuous power capable of being so produced is a joint function of the volume of tidal inflow impounded in the headwater basin and the minimum operable head under which the water, so stored, can be passed through the turbines under neap-tide conditions, and that, in turn, the operable head, as predetermined by considerations of rational turbine design, capacity and efficiency, is a joint function of the headwater and inner tailwater level, as adjusted and co-ordinated during the critical period of operation intervening between the closure of the outlet gates, during the rise of the tide, and their subsequent opening during the succeeding ebb.

The calculations relative to obtainable operating head and power capacity involved several independent variables which made a rational solution impossible, so that the answer could be obtained only through the agency of a tedious and somewhat complicated mathematical process of trial and error between maximum and minimum limits. The details of this study are covered at length in later sections of this report.

## 3. SPECIAL AND INCIDENTAL FEATURES

The most obvious and special feature in connection with this study is, of course, the fundamental problem of creating an operable and continuously-sustained head from the oscillating and constantly-varying tide levels associated with the Petitcodiac site. The study of this phase of the problem is covered at length in a later section of this report, and requires no further comment at this point.

Another special consideration peculiar to the site is the matter of deterioration of metal parts permanently or periodically submerged in sea water.

Metal parts periodically submerged in sea water offer no particular difficulties. The systematic testing programs carried out over a number of years by various interested agencies have proved that cheap and reliable protection from corrosion may be obtained by the careful and regular application of anti-fouling or other suitable paints. Therefore, any elements of a hydro-electric installation that can be periodically removed from the salt water, carefully inspected and painted, will cost little more from a maintenance standpoint than the same elements which are periodically submerged in fresh water. Such items include lock gates, sluice gates and racks.

Metal parts permanently submerged in sea water and forming part of a hydro-electric development, such as discussed herein, may be divided into three main groups or classes:—

(*a*) Steel sheet piling as later proposed herein for the central core section of the rock-fill dams.

(*b*) Metal check liners for emergency gates and racks.

(*c*) Embedded portions of the hydraulic turbines and metal-lined sections of the draft tubes.

For some years past a small percentage of copper has been added to carbon steel in the rolling process of steel sheet piling with the idea of providing additional resistance to corrosion. Actual tests have already proven that no substantial resistance to corrosion is provided by the addition of copper. At this date the rolling of corrosion-resistant alloy steel into steel sheet piling has not been considered practical, particularly from the standpoint of cost. In the rock-fill dams at the Petitcodiac site, steel sheet piles 80 feet in length are necessary and these would be carbon steel piles. Such piles must have sufficient web thickness to withstand the heavy battering which they would be subjected to during the driving process. After being driven they will be in contact with salt water, but the water will contain very little entrained air. Therefore, it is hereinafter considered that the average rate of pile corrosion will be approximately ·0075 inches per year per side of pile, or ·015 inches per year per pile total. On this basis the total corrosive effect measured in the web thickness of the pile would amount to ·45 inches over a 30-year period. The web thickness of the piles provided in the estimates herein is ·875 inches. Long before the 30-year period has elapsed, all interstices in the stones forming the rock fill should be sealed by the silt which will obviously be deposited on the slopes of the dam by the incoming and outgoing tides, and the steel sheet piling will cease to be a necessary or integral portion of the dam structures.

Obviously, the same argument will not apply to the metal check liners provided for the emergency gates and racks and some type of corrosion-resistant metal must be provided for their manufacture. The cost of such corrosive-resistant sections has been included in the estimates which follow hereafter.

Portions of the turbines and the metal portions of the draft tubes may be said to be permanently submerged; that is, they may not be withdrawn into the open for inspection and painting. However, the design of the power-house structure will be such that the embedded parts and the draft tubes of any unit may be unwatered at any time for inspection and maintenance. During such a period careful inspection of these various submerged parts may be made, and if it is found that periodical painting has not given complete protection, any metal lost by corrosion may be built up by the very expert and efficient present-day processes of welding or metal spraying.

The final consideration under this head is the critical construction problem relative to the final closure of the rock-fill dams across the two estuaries, against tidal currents of progressively increasing velocity, which reverse themselves in

each cycle. The method of solution will appear in a later section of the report covering "Construction Procedure and Sequence of Operations".

The more important items classified as incidental are covered below under sub-headings as indicated.

**Navigation.**—The Petitcodiac River is a navigable stream at certain stages of the tide, and tankers and ocean-going steamships of limited tonnage dock more-or-less regularly at Hillsborough and Moncton. The observed practice has been for vessels to anchor off Hopewell Cape and to await there a favourable tide condition before proceeding further up-river.

The proposed construction of any barrier in the Petitcodiac River between Hopewell Cape and Fort Folly Point necessitates the incorporation therein of locking facilities, to such extent that, in general, navigation will not be inconvenienced or unduly delayed. Due to topographical and foundation conditions, and to the fixed location of other main elements of the permanent structure layout, it would be necessary to construct any locking facilities close to the Hopewell Cape shore, where ledge rock provides an unyielding foundation. A detailed description of the lock and adjacent docks is set out under the section dealing with "Design of Permanent Works".

The construction of a dam at the above location will have a most favourable result upstream from the dam and lock, insofar as navigation is concerned because a permanent basin will thus have been established, and once through the lock vessels may pursue their course up or down river, without hazard or hindrance. The lock floor will be constructed at such elevation that vessels may enter during the period from one hour before high tide up to one hour after high tide.

While the Memramcook River may be said to be a navigable stream at certain upper ranges of tide, present-day shipping is limited to fishing-boats of the motor-boat and small sail-boat class, which do not require the construction of a lock in the barrier across the Memramcook estuary. To maintain such facilities as are necessary in the premises, the estimated capital costs set out hereafter will cover the construction of a wharf downstream from the outlet gate structure, including the cost of extending the road, now leading to Coles Head Wharf, westward to the new proposed wharf.

**Rail Access.**—The construction of a standard gauge railway connecting with the main line of the Canadian National Railways will be one of the first major operations of the ancillary construction program. The lack of suitable docking facilities and the practical impossibility of constructing such facilities at Fort Folly Point mean that heavy equipment, both temporary and permanent, cannot be water-shipped to the site. Therefore, the matter of rail access is of prime importance and necessity. Quite possibly it will also be necessary to transport all concrete aggregates for some considerable distance by rail.

A reconnaissance of the territory adjacent to Fort Folly Point would indicate that the proper junction of the construction railway with the main line of the Canadian National Railways would be at or near Gayton, some 17 miles east of Moncton. The projected line would almost immediately cross the Memramcook River, a very narrow crossing at this location, and would follow the west shore of the Memramcook for a distance of about 15 miles to the power-house site at Fort Folly Point.

The cost of constructing this railway is included in the section of the report dealing with estimated capital costs.

**Flowage.**—A reconnaissance survey of the dyked marsh lands adjacent to the Petitcodiac River, made in the autumn of 1944, indicates that the marginal boundaries of the Petitcodiac high-level basin will include some 1,700 acres of

dyked marsh lands. These dyked areas are in various states of productivity. The survey indicated their condition as follows:—

| | |
|---|---|
| Dyked areas in good or fair condition | 1,432 acres |
| " " subject to considerable repair | 189 " |
| " " of no practical value | 101 " |

Each individual dyked area is usually provided with an outlet gate, so operated that the tidal salt water may not enter at any tide range, but allowing the fresh surface water and possibly a portion of the fresh ground water to flow out into the river at certain stages of each ebb tide. The elevation of the inverts of these outlet gates vary from as low as Elevation $-7\cdot0$ to as high as Elevation $+12\cdot0$ and possibly higher.

The elevation of the marsh lands within the dykes, in general, may be said to be $+20\cdot0$ or thereabouts.

In the normal course of operation of the Petitcodiac high-level basin, during all but the extreme high tidal periods, there should be, periodically, sufficient head differential between the water on the marsh side of the dyke and the water in the high-level basin, so that the outlet gates may function without trouble. During extreme high tidal periods, however, the head differential may decrease to the point that certain outlet gates, as constructed, cannot function. To take care of such eventualities, an allowance has been included in the estimates of capital cost set out later herein.

The marsh lands tributary to the Memramcook basin will not be subject to the same flooding hazards as those tributary to the Petitcodiac basin, because the Memramcook basin will be operated as the low-level basin, and the water surface therein will be at a permanently lower level than it is at present under the higher ranges of tide. Any interference with dyked marsh flowage in this basin then will be only in connection with lands upon which dredging is necessary to enlarge and deepen the basin. In this case, also, an appropriate allowance has been included in the costs of dredging which form part of the estimates of capital cost herein.

**Sewage Disposal.**—At the present time all sewage from the City of Moncton and its immediate suburbs is discharged directly into the Petitcodiac River or into Halls Creek, its tributary stream east of the city.

The full range of the tide is not experienced at Moncton. The city is situated some 20 miles up from the mouth of the river, and therefore "Low Water" is cut off by the river bed slope. This fortuitous circumstance means that all sewage is successfully and swiftly carried down river on the ebb tide twice daily, and that all sewer outlets and low-lying sewer sections are completely flushed out twice daily at the higher stages of the incoming tides.

For purposes of record the elevation of certain main sewer outlets of the city proper are listed hereunder:—

| Sewer Location | City of Moncton Elevation | Geodetic Elevation |
|---|---|---|
| West C.N.R. Yards | 5·08 | 12·24 |
| Foundry Street | 9·13 | 16·29 |
| Robinson Street | 6·71 | 13·87 |
| West Market Street | 4·30 | 11·46 |
| Downing Street | 9·25 | 16·41 |
| Winter Street | 12·00 | 19·16 |
| Steadman Street | 7·41 | 14·57 |
| Church Street (Extension) | 8·80 | 15·96 |
| West of Church Street (Extension) | 8·00 | 15·16 |
| Robinson Street—North | 9·82 | 16·98 |
| Highfield Street (Extension) | 11·00 | 18·16 |
| North Street | 9·96 | 17·12 |

Comparatively, the operating level of the Petitcodiac basin under various tide ranges, and with the units operating to produce constant power at 100 per cent load-factor, is listed below:—

| Top Elevation of Tide | Low Elevation of Basin |
|---|---|
| 22·0 | 14·5 |
| 18·0 | 10·8 |
| 15·0 | 8·0 |
| 12·0 | 5·0 |

Comparing the operating low-level elevation of the basin with the elevation of the various sewer outlets, it will be seen that even during a high tidal period the majority of the sewer outlets will drain freely into the high-level basin, so that the cost of constructing pumping stations at those outlets which will not drain freely would not be excessive when considered as an integral part of the project herein contemplated.

Unfortunately, the fact that sewage may still be discharged at small cost into the Petitcodiac basin does not constitute the answer to this problem as a whole.

The construction of a barrier across the Petitcodiac River at Hopewell Cape, and the utilization of the river as an operating high-level basin, will mean that the Petitcodiac River, at present a muddy, fast-flowing stream, will be transformed into a more-or-less clear water lake, and during periods of draw-down there will be a definite tendency for sewage to be carried downstream, well below the environs of the City of Moncton. In the wider reaches of the river below, however, there will be a decided tendency for the current to become sluggish, and a decided probability that sewage will alternately move down and up stream, perhaps several times before ultimately being discharged into Shepody Bay.

The only practical answer to such an operating condition is the construction of a complete sewage disposal plant for the City of Moncton and its suburbs, along with the construction of such main intercepting sewers as may be necessary. The costs of such remedial works are included in the main estimates which appear hereafter.

**Wind Effects.**—It is, of course, evident in principle that a gale coming in from the sea will cause an additional increment in the culminating level of inflow, and that a gale in the opposite sense will have the opposite effect, in both cases in a degree proportional to the velocity of the wind. Furthermore, the effect of these winds on the tide levels will not only be affected by their velocity, but by the quarter from which they come, thus creating a condition of fortuitous circumstance which is not susceptible of any rational solution so far as the actual effect on the operating head is concerned.

Some conception of the conditions which can result from a wind of hurricane proportions is obtainable from the records of the so-called "Saxby Gale".

The Saxby Gale was apparently of more-or-less hurricane proportions reaching a peak early on October 5, 1869. Its peak was coincident with the highest tide ever recorded in the Upper Bay of Fundy and known as the "Saxby Tide".

The following significant data is quoted from "Tides at the Head of the Bay of Fundy", by W. L. Dawson, D.Sc.:—

"This tide occurred at 1 a.m. on October 5, 1869. The moon's position at this date was as follows:—New Moon, October 5 at 10h. 19m. Perigee, October 5 at 3h. Moon on Equator, October 5 at 8h. There was thus no diurnal

inequality at the time, to add to the height of the tide. The exceptional rise was due to a gale at the perigee springs."

A comparison of the Saxby Tide with other extreme high tidal stages is given below:—

Minas Basin .........Saxby Tide 3·05 feet higher than next recorded extreme stage.
Cumberland Basin ....Saxby Tide 3·17 feet higher than next recorded extreme stage.
Moncton ............Saxby Tide 6·45 feet higher than next recorded extreme stage.

From the above data it would seem evident that the high tidal stage at Moncton was accentuated to a much greater degree than lower down in the Bay of Fundy. The elevation of the Saxby Tide at Moncton is recorded to be 32·26—Geodetic Datum. By interpolation it would seem logical that the Saxby Tide should reach an elevation of from 28·0 to 29·0 at Hopewell Cape. To take care of any future abnormal combination of tide and wind, the crest of the dam structures adjacent to Fort Folly Point has been fixed herein at Elevation 36·0—Geodetic Datum.

From the above facts it is, of course, evident that the barrier structures across the two estuaries must have sufficient stability, and a sufficient margin of free board, to safely withstand the impact of tides which are under maximum stress from winds of hurricane proportions, but the secondary effect of these winds upon operating head and upon the normal daily regimen of gate operation and control can be determined only as a result of actual operating experience.

**Ice.**—It will be necessary to protect the barrier structures against injury from masses of ice moving to and fro in the estuaries with the ebb and flow of the tide, but it is not anticipated that any winter conditions, which may obtain, will seriously affect plant operation, or the production of power. Any ice which becomes lodged in the head-water basin may be expected to remain there until it disintegrates under the influence of temperature, and any portion of it which may pass through the turbines will not be sufficient in quantity to affect the functioning or operation of the tailwater outlet gates.

**Fresh Water Supply.**—There is no indication that fresh water will be available in any adequate quantity at or near the power-house site, but as regards cooling-water requirements, the design herein submitted provides for the use of the immediately-available salt water supply. The comparatively small amount of fresh water which will be required for domestic and other incidental purposes, at or near the site, is covered by provision in the estimates for a salt water evaporator and such storage, cooling and hardening facilities as may be required.

**Silt.**—Through the courtesy, and with the assistance of the Dean of Engineering of Mount Allison University and his laboratory staff a determination was made of the silt content of the tidal water. The analysis made of the water is shown in the attached Appendix No. 1, and indicates that the silt problem associated with this project does not constitute a hazard either to operation or to maintenance.

**Weight of Water.**—Again, through the good offices of the Dean of Engineering and the staff of Mount Allison University, a determination was made of the weight of the water which would pass through the turbines for the production of power. The results of this determination are shown in the attached Appendix No. 2. As compared with the nominal weight of 62·4 pounds per cubic-foot, which is ordinarily used in power calculations, it will be seen that

the estuary water has a weight of 63·7 pounds per cubic-foot with silt eliminated, which in turn will have the effect of increasing the energy potential of the estuary water, as compared with fresh water, by about 2 per cent.

## 4. SURVEYS AND INVESTIGATIONS

**Subsurface Exploration—Memramcook and Petitcodiac Dam Sites.**—The possibility of developing power from the tidal waters of the Bay of Fundy has been given some considerable thought over a period of many years.

In the early and middle twenties, various preliminary reports were prepared on this subject, and official interest was such that, in 1924, the Department of Public Works of Canada instituted a program of drilling at the point where the Memramcook and Petitcodiac Rivers flow into Shepody Bay.

During the period July 28 to August 23, 1924, under the direction of Mr. H. M. Davy, of the Test Borings Branch, Department of Public Works, a series of 39 test holes was drilled in this general location. Of this number 23 were located in the Petitcodiac River area, 14 were located in the Memramcook area, and the remaining 2 were located some considerable distance up the Petitcodiac River adjacent to the Village of Hillsborough. Later on, during the period September 6 to October 7, 1927, an additional 15 holes were drilled in the same general area. Of these 15 holes, 11 were drilled in the Petitcodiac River area, and 4 in the Memramcook area.

The results of these two drilling campaigns are illustrated on a plan issued by the Department of Public Works of Canada, entitled "Location and Detail of Test Borings at Shepody Bay, New Brunswick". On this plan is indicated the complete log of each hole drilled, showing in detail the various classifications and depths of materials through which the drill passed before reaching the ledge rock stratum below.

The information, in general, assembled by the Department of Public Works was so complete in character, and so almost exact in location, that it was not considered necessary to carry on further substratum exploration for the purpose of this report. Therefore this information, as set out in the plan above referred to, forms the basis of the structural layout and cost estimates which follow, in so far as the underwater portions of the two rock-fill dam sites are concerned.

The plan prepared by the Department of Public Works, in connection with the location and detail of the test borings, is not reproduced in this report, nor are the detail logs of the various holes drilled, but the elevation of the river bottom, and of the rock stratum beneath, is shown graphically in contour form on Plate No. 13.

No additional exploratory drilling should be necessary in connection with the final location of the lock, inlet gate structure and power-house. Subsurface surveys made by the Department of Public Works in the general area of the outlet gate structure may have to be amplified prior to actual construction, and allowance has been made in the estimates for this contingency.

**Exploration of Memramcook River Basin.**—As the Memramcook basin would act as the low-level basin in the operation of the hydro-electric station, as described generally under Section No. 1 of this report, it was most essential that a complete survey be made of the Memramcook River bottom and its marginal marsh lands. Such a survey would necessarily extend from Fort Folly Point to a location somewhat beyond the Village of Memramcook. In August, 1944, therefore a survey party was assembled by H. G. Acres & Company, and the exploration of the Memramcook River basin was its first major undertaking.

An experienced hydro-electric construction engineer was sent to act in resident charge of the survey party, having as his assistant a field engineer of long experience. In addition, an experienced instrument man was sent from H. G.

Acres & Company's Head Office. A staff of 10 was employed on this survey work, the remaining members consisting of transitmen, draftsmen, rodmen, and chainmen recruited from the local municipalities. The party continued work from August, 1944, up to and including December 24, 1944.

First of all, the Memramcook basin, including all marshes and dyke lands, was traversed and levelled, and the main traverse lines were tied into the main line of the Canadian National Railways for checking purposes. This work, in general, extended from Fort Folly Point to beyond the Village of Memramcook.

Upon completion of the ground work above described, the Memramcook River proper was cross-sectioned at various intervals, from Fort Folly Point upstream as far as the river had any significant width.

It was necessary to do most of the sounding work from motor-boats, and the wider sections of the Memramcook River were sounded with considerable hazard and difficulty, due to high wind and adverse weather conditions.

With the information and data secured from this most complete survey of the area, it has been possible to compute the area of the existing river basin at any desired elevation, and also to compute the volume of dredging necessary under any stage of power production contemplated.

Survey work in the Memramcook River area is shown graphically on Plates Nos. B1 to B11, inclusive.

**Surveys at Fort Folly Point.**—Upon completon of the survey work in the Memramcook basin, Fort Folly Point was thoroughly and carefully cross-sectioned.

First of all, lines for an encompassing grid were cut through the bush for a distance of some three-quarters of a mile up the shorelines of both the Memramcook and Petitcodiac Rivers. A series of levels was then run over each of the lines comprising the grid, and accurate contours thus obtained for the whole Fort Folly Point area. As this area is a mass of ledge rock with just enough overburden on which to allow tree growth, it was not necessary to carry on any subsurface drilling.

**Surveys Adjacent to Hopewell Cape.**—On the west side of the Petitcodiac River surveys were extended upstream and downstream from the existing Government Wharf, and the area accurately cross-sectioned.

A gauge was established at the end of the wharf and read daily during the period September 22 to October 28, 1944, inclusive.

**Surveys Adjacent to Coles Head Wharf.**—Similar surveys were carried out on the east side of the Memramcook River, downstream from Coles Head Wharf. This area was also carefully cross-sectioned, as it is in this location that it is proposed to install the outlet gate structure.

**Exploration of Petitcodiac River Basin.**—The information required in the Petitcodiac River basin was somewhat different from that in the Memramcook basin. As the Petitcodiac basin would be operated as a high-level basin only, there will consequently be a minimum elevation, during an extreme neap tide period, below which the water level would never be drawn. Therefore it would not be necessary to actually cross-section the Petitcodiac River below this minimum operating elevation.

As the Petitcodiac basin is some 35 miles long, involving many stretches of fast water and treacherous bottom, it was decided that much of the information required could be obtained more accurately, more conveniently, and more safely by means of an aerial survey; the aerial photographs to be taken at or near the low range of tide.

A contract was accordingly entered into with the Canadian Pacific Air Lines Limited to make the necessary aerial photographs and prepare from

these a contour plan which would be in sufficient detail to permit the calculation of the basin area at given elevations.

Canadian Pacific Air Lines Limited had their 'plane at Moncton Airport on October 23, 1944, but experienced several weeks of extremely unfavourable weather, and it was not until November 27 that flying conditions were such that a set of low-level aerial photographs was obtained on both banks of the Petitcodiac River. Following this date the ground survey party established a considerable number of control points in connection with the aerial survey above mentioned. In order to establish the proper control points, it was necessary to run almost continuous lines of levels along both sides of the Petitcodiac River from Hopewell Cape to well above Moncton. The field survey party completed this phase of the work on or about December 24, 1944.

Canadian Pacific Air Lines Limited continued into the winter months preparing the necessary contour plans, which were finally completed and delivered on or about March 31, 1945. The results of this survey are illustrated on Plates Nos. A1 to A5, inclusive.

**Investigations.**—Supplementary to the making of the actual surveys above described, and directly associated with them, were (1) investigations made with reference to the corrosion of metals in sea water, and (2) the research and investigation made in connection with the Passamaquoddy Tidal Project on the coast of Maine.

In the early summer of 1944, Mr. H. E. Barnett, Consulting Engineer of H. G. Acres & Company, through the courtesy of the International Nickel Company, visited the testing station at Kure Beach, in North Carolina, which is maintained by the latter company. For the past 9 years, at Kure Beach, the International Nickel Company has carried on extensive investigations in connection with the corrosive effect of sea water and sea air on a great many metals and alloys. Every year, during the month of May, engineers interested in the results of these investigations are invited to Kure Beach by the International Nickel Company, and during this visiting period a certain definite number of specimens are removed from the sea water, carefully inspected, and the corrosive effect of their immersion in sea water measured and recorded. A great deal of valuable information has thus been accumulated during this term of years.

The several paragraphs set out previously under Section No. 3 of this report, dealing specifically with the corrosive effect on metal structures associated with the Petitcodiac project, are based upon the information received during this visit to Kure Beach, and upon subsequent correspondence with the International Nickel Company dealing specifically with Petitcodiac problems.

In the autumn of 1944, arrangements were made by the Controller of the Dominion Water and Power Bureau, Ottawa, with the Acting Chief of Engineers, United States Army, for representatives from H. G. Acres & Company to visit the District Engineer at Boston, Mass., in connection with the examination of data recorded during the Passamaquoddy investigation and initial construction, in the early 1930's.

In November, 1944, Messrs. A. W. F. McQueen and H. E. Barnett, Hydraulic and Construction Engineers, respectively, of H. G. Acres & Company, spent 3 days in the office of the District Engineer at Boston, where every facility was accorded them to study the various reports which had been prepared from time to time on the Passamaquoddy project, and to discuss with their engineers the actual problems encountered during the period of investigation and during the short period that construction was actually carried on. The results of this visit were most helpful in a general way, and specifically so in connection with the model studies that had been made with reference to the sinkage of rock-fill dams in material which closely approximates that over-

lying the rock stratum in the Memramcook and Petitcodiac estuaries. These studies are referred to later in Section No. 8, dealing with the "Design of Permanent Works".

## 5. AVAILABLE RANGE OF OPERATING HEAD

**Tides in General.**—All along the coasts of the world there is a continuous rise and fall of the ocean level, usually without secondary movements, the period of which is approximately 12 hours and 25 minutes. There are thus two times of high water and two of low water nearly every day. The two high waters do not usually rise to the same elevation, though in some cases, as in the Bay of Fundy, the difference is not very pronounced. It is this rise and fall of the water level, and the effects produced by it, that are known generally as the "tides".

At any point on an open coast the average level of the water surface is called "mean sea level". Once established, this level remains constant. In other words, on the average, the water rises as much above mean sea level as it falls below it.

Wherever there is an estuary or inlet with narrowing shores or similar topographical conformation, the rise of the ocean water causes a flow to set in up the estuary or inlet. This flow is known as the "flood". The corresponding outward flow of the water during the fall of the tide is known as the "ebb". Slack water occurs at the top of the tide and at the bottom. These movements are known as "tidal streams". It is important to distinguish clearly this aspect of the tides from the vertical rise and fall which are the tide proper. The direction and velocity of the tidal streams at the site of the proposed power development will have a decided influence on the design and construction of all permanent works.

The range of the tide is defined as the difference in level between successive high and low waters, or between successive low and high waters. It varies from day to day, usually rising to a maximum twice every lunar month. These high tides are know as "spring tides". About midway between each spring tide, the range reaches a minimum value, and these low ranges are called "neap tides".

The tides are caused by the attraction of the moon and the sun on the ocean and on the earth. Because it is so much nearer the earth than the sun, the moon exerts a much greater influence on the tides than does the sun, despite its comparatively small size. Hence a knowledge of the relative movements of the sun, moon, and earth is necessary for an understanding of the tides, and particularly of the variations in the range which take place, in a systematic manner, over a long period of time.

When the earth, moon, and sun are in the same straight line, which happens about twice a month, the attractions of the moon and sun act together and the range of the tide is much greater than when the moon, earth and sun are in quadrature. Thus, the time of the new moon and the full moon produces the spring tides; and when the moon is at its quarters, the neap tides occur. The time from new moon to new moon is known as the "synodic month". Because of the moon's perturbations, that is, the irregularities in its movement due to the sun's attraction, the exact time of the synodic month varies but its average value is 29·53 days.

The path of the moon as it revolves around the earth is not circular but elliptical. Moreover, the earth is not in the centre of the ellipse, as might be supposed, but is off centre. Therefore, during each revolution of the moon around the earth there is one point, and one only, where the moon is nearest the earth. This is known as "perigee". The point where the moon is farthest from the earth is called "apogee". The period of time from perigee to perigee is the anomalistic month. Its average length is 27·55 days.

The position of the sun in the heavens, at its zenith, varies from day to day. In the northern hemisphere it is higher in the summer than in the winter. This is due to the well-known fact that the plane of the equator does not coincide with the plane of the earth's orbit. This angular position of the sun with respect to the earth's equator is called its "declination". In a precisely similar way, the position of the moon in the heavens varies. The period of time that elapses between the successive times that the sun crosses the equator in the same direction is, of course, about one year, whereas for the moon it is about one month, or more precisely, 27·32 days. This period is known as the "tropical month".

These three movements of the moon are its chief ones and all have an effect on the tides. However, at any particular place on the earth, one of them has a dominating influence, the effect of the other two movements being more-or-less secondary. There are, therefore, synodic, anomalistic and declinational type tides.

**Range of the Tide at Hopewell Cape.**—In any tidal power scheme it is the range of the tide which determines the head available for actuating the turbines. The extent of the range and its variation are therefore matters of prime importance to this investigation.

The tides of the Bay of Fundy are of the anomalistic type. The maximum range in any month occurs, therefore, when the moon is at perigee and not at the new and full moon.

Before discussing the tidal range at the mouth of the Petitcodiac River, attention should first be directed toward the data upon which this discussion must be based. During the summer of 1919 a tidal recording gauge was established on the new wharf at Hopewell Cape. A continuous graphic record of the rise and fall of the tide at this point was obtained from July 3 to October 1. In the autumn of 1944 a staff gauge was established at the same place, and sufficient readings taken to obtain the elevation of the daylight high and low tides for the period September 22 to October 17. These two sets of records are not sufficiently long in themselves to establish the limits of the tidal range nor its variations. Fortunately, the tide tables at Saint John, New Brunswick, which are available from 1904 up to the present time, can be used as a reference. The ratio of the rise above and of the fall below mean sea level, to the corresponding figure derived from the tide tables at Saint John, was computed for each of the recorded tides at Hopewell Cape. The average of each of these two sets of figures gives a factor for use with the Saint John tide tables to obtain a derived record at Hopewell Cape for any period of time up to a maximum of 41 years.

A reproduction of a typical tidal chart taken at Hopewell Cape is included in this report as Plate No. 3. The period covered by this chart is the full week of August 1 to August 7, inclusive, 1919. Perigee occurred on July 23 of that year and apogee on August 4. First quarter was on August 3 and full moon on August 11. Low tides were therefore due to occur during the week in question and the chart shows that this is what actually happened since the range falls below 28 feet, which is quite a low tide indeed. It should be noted here that the chart ordinates must be doubled to give the true scale in feet.

It is clear from this chart, also, that the tides are extremely regular, and that the difference in range between the two daily tides is quite small. This inequality in the range is due to the moon's declination, and from this and other features of the tides at Hopewell Cape, it may safely be assumed that the influence of the moon's declination is a comparatively minor one. Nevertheless, as will be seen later, this is a point of some importance in the actual derivation of a duration curve of tidal ranges. It follows, therefore, that at Hopewell Cape the major influences are the distance of the moon from the earth, and the moon's phases, with the former having the greater effect of the two.

As previously mentioned, each of the three leading movements of the moon has its effects on the tides. The full variation in the range of the tides will

therefore occur during one complete cycle of the moon's movements in relation both to the earth and the sun. This complete cycle occupies 18 years and 11 days. In other words, every 18 years and 11 days the sun, moon, and earth return to the same relative position in the heavens, and at all times within this period, the relative position of these three bodies is different.

Given a tabulation of tidal heights for 18 years, it would be possible to construct, directly, a duration curve of tidal ranges. However, in this case, it was necessary to derive the tidal heights by reducing the tabular values of high and low water at Saint John Harbour (four per day), first to mean sea level, then to the corresponding range at Hopewell Cape, followed by the calculation and averaging of successive pairs of ranges. The average of each two successive ranges, referred to one high water, must be used since the power potentiality of any tide is affected by the range on the falling side, as well as on the rising side. The work involved in making such a series of computations as above described, for over 18 years, is so considerable that a practical approximation to this result was worked out, as explained below, in order to save time and labour.

As previously noted, the tides of the Bay of Fundy are influenced chiefly by the distance of the moon from the earth and by the moon's phases, therefore, the cycle of the anomalistic and synodic months represents a first approximation to the desired result. Using average values for these two months, the length of this cycle is found to be very close to 1,622½ days, or a little under 4½ years. A duration curve of range for such a period from July 30, 1940, to December 31, 1944, was computed and plotted.

The next step, or modification of this curve to adjust it for the effect of the moon's declination, involved the computation of the maximum tide in each year, the minimum tide in each year and the yearly average of each of the two monthly high tides and the two monthly low tides for the 18-year period 1927 to 1944, which included, of course, the 4½ years of the preliminary duration curve. The plotting of these results is shown on Plate No. 4. It is clear from the plate that the period used for the first approximation, namely, July 30, 1940, to December 31, 1944, is not only one of higher-than-average tides, but contains the highest tide of the entire 18-year cycle. The entire duration curve, with the exception of the upper limit, must therefore be modified downward. The figures derived in the manner just described showed how this should be done and the modified and final duration curve is given on Plate No. 5.

It is interesting to note that the highest spring tide for the 18-year period of Plate No. 4 occurred in 1940 with a range of 52·2 feet, and the lowest spring tide in 1929 with a range of 46·5 feet. The highest neap tide also occurred in 1940. Its range was 27·4 feet. On the other hand, the lowest neap tide of 1929 had a range of 22·2 feet, while those of 1930, 1931, 1932, and 1933 had ranges of 21·9, 22·2, 21·1, and 21·4 feet, respectively. All three curves on this chart indicate clearly that there is a gradual change in tidal range over the entire 18-year cycle, with but one general high period and but one general low period. It is also equally clear that the maximum ranges do not follow each other in any definitely regular progression, and the same is true of the average and minimum ranges. As previously explained, these curves are based entirely on the Tide Tables and therefore on predictions. Accordingly, no explanation of these irregularities is given here since they are due to astronomic effects. The actual tides will vary from the predicted figures by small amounts, particularly during periods of heavy winds. These more-or-less fortuitous variations, however, will tend to even out and will have no significance as related to either the specific or general conclusions reached in this report.

The duration curve of Plate No. 5 shows that the extreme maximum range is 52·2 feet and that the extreme minimum range is 21·1 feet. The ranges associated with the high tides have but limited significance as far as the development of power is concerned since, as will be explained later, no attempt

has been made in the studies to extract their full power potential, but they are of prime importance in determining the elevation of the crest of the tide retaining structures. This matter of the maximum tides is discussed more fully elsewhere, in connection with the description of the Saxby Tide, and also in connection with the setting of the sills of the outlet gates.

It is interesting to note, also, that the curve shows the following:—

| | | | | | |
|---|---|---|---|---|---|
| For 98% | of the time | the range is | 23·5 | feet | or greater. |
| For 95% | " | " | 25·0 | " | " |
| For 90% | " | " | 26·7 | " | " |
| For 85% | " | " | 28·1 | " | " |
| For 50% | " | " | 34·3 | " | " |

The mean range, from the curve, is 35·15 feet, as compared with the 34·3 feet equalled or exceeded for 50 per cent of the time. The curve is therefore skewed toward the high ranges. This is also evident from an inspection of Plate No. 5. This is a characteristic of the anomalistic type of tide and explanations for it can be offered based on the known movements of the moon. Such an explanation is beyond the scope of this report. This feature does mean, however, that the variation in the range is greater than would occur with the more usual synodic type of tide.

## 6. DESIGN AND CHARACTERISTICS OF TURBINES AND CONTROL GATES

**Selection of Turbines.**—The matter of the selection of a suitable combination of turbine and generator for a tidal power plant presents problems which are not encountered in ordinary hydro-electric practice. They may be resolved into four main divisions, viz., the determination of (*a*) the proper type of turbine, (*b*) the most economical size of turbine, (*c*) the best speed and blade angle, and (*d*) the power of the generator. This last point, the power of the generator, arises from the fact that the turbine, at full gate, will have a wide range in power output because of the great variation in head. The head not only varies considerably during each tide due to the changing level in headwater and tailwater, as previously explained, and as will be demonstrated graphically later, but it also varies with the tidal range, from month to month and from year to year. Obviously, it would be uneconomical to select a generator to match the power which the turbine could produce at maximum head during the very high tidal ranges, since this would represent use at or near maximum capacity for only a small percentage of the total time. How the final selection was made is discussed hereunder.

For the low heads found in a tidal power development there are two turbine types available, viz., the fixed-blade propeller and the adjustable-blade propeller or Kaplan turbine. Besides being more expensive, the Kaplan turbine is mechanically more complex than the simple fixed-blade machine, and, as a result, it is more vulnerable than the latter to the hazard of corrosive action by sea water to which most construction metals are subject. Hence, the advisability of simplification of turbine design wherever possible in the present instance.

A second point of comparison between these two types relates to the rather unique method of operation which is possible in a large tidal power plant. Because of the low head necessarily associated with such plants, the turbines will be large physically but comparatively small in power capacity. Hence, many units will be required to develop a block of power of any appreciable magnitude. If the maximum possible amount of firm power, at 100 per cent load-factor, is being produced, then the turbine gates must be practically wide open at the lowest head which occurs during the lowest tides. The reason for limiting this statement to the production of firm power at 100 per cent load-factor will appear later on in the discussion. Whenever the head is greater than the minimum, the

operating staff has the choice of running all units at partial gate or the necessary number at or near full gate. The latter alternative involves shutting down units as the head increases and starting them up again as the head decreases, an operation that can be performed by automatic instruments. The former choice involves the use of Kaplan units whereas the latter can be accomplished with fixed-blade machines, with very little or no difference in efficiency between the two methods of operation.

In fresh-water hydro-electric plants the Kaplan turbine has found its chief usefulness where operation at high efficiency is required over a wide range of gate openings for a considerable portion of the time. At such plants the variation in head is usually not great so that the load-factor is comparatively low. In the discussion of the power studies, Section 9, it will be shown that in a tidal power plant the head must vary over a wide range if the maximum amount of power, or of energy, is to be extracted from the tides. Moreover, in the proposed Petitcodiac plant the number of units is so great that operation at maximum efficiency of the necessary number to give the desired output can be arranged. Under these conditions, therefore, the Kaplan turbine has no advantage over the fixed-blade propeller machine.

On the basis of simplicity and ruggedness of construction, and hence economy in capital cost, and of relative efficiency, the choice as between the Kaplan turbine and the fixed-blade propeller turbine falls definitely on the latter type.

Plate No. 6 shows the relation between efficiency and head for a fixed-blade, propeller type turbine operating at the gate opening corresponding to that giving best efficiency. This curve demonstrates that a high efficiency can be maintained by a unit of this type over a very wide range of heads. The break in the curve at 26 feet net head is due to the fact that beyond this point the output per hydro-electric unit has been limited in this particular case by the selection of the generator capacity. Hence, as the head increases, the turbine gates must move in the closing direction in order to keep the turbine output at a constant value corresponding with the maximum generator capacity, and the efficiency curve takes the unusual shape shown on the Plate.

The most economic size for the units is determined by several factors. One of these relates to the effect of the number of units on the construction quantities. Many small units require a long power-house, a wide entrance and exit channel, and comparatively shallow excavation. Since the location of the outer end of the power-house is fixed by the topographical features of the site and their relation to construction requirements, as explained elsewhere, a long power-house means that the inner end would extend well into Fort Folly Point and that the excavation for the entrance channel must of necessity cut through a higher section of the Point than would otherwise be necessary. Heavy excavation quantities would result. On the other hand, a smaller number of larger units would require a shorter power-house, a narrower forebay and tailrace, but much deeper excavation.

Another factor has to do with the effect of the size of the individual machines on the total cost for a given horse-power installation. A large number of small turbines involves higher machining and handling costs than for the equivalent number of larger units.

The cost of the turbines is practically unaffected by the speed of rotation. This is not true of the generators, the price of which, for a given maximum generating capacity, decreases as the speed increases. Since a small turbine allows use of a higher speed, generator costs must be considered before the final selection can be made.

The efficiency of a fixed-blade, propeller type turbine, working under a wide range of heads, varies considerably with the speed. There is also a variation with the pitch of the blade, but a pitch angle can be selected that is best suited to the range of gate openings under which the unit will operate

and which will give maximum efficiency for this condition. Similarly, a speed can be selected which will give the highest efficiency over a particular range of heads. Selection of this range is based upon the consideration that the maximum possible amount of power must be developed from the low tides. Furthermore, the highest efficiency of generation must be obtained when the power output from the station is at or near its natural minimum. If the efficiency drops during medium and high tides, this is a matter of small concern since there will then be an excess of power.

The best angle and best speed vary with the designs offered by the different manufacturers. These points were fully discussed with the Canadian turbine manufacturers and the curve sheets and power studies of this report are based on the best one of the present available designs. Each of these designs had been developed to cover a range of heads selected on the basis of fresh water hydro-electric power plant practice. It is probable that these designs could be somewhat improved by experimentation and study based upon the exact requirements for Petitcodiac, and while a decision to proceed with construction would warrant such further study, the possible betterments realized would not affect the substance of this report in any noticeable degree.

Preliminary studies, taking into account all of the above factors, indicated that a turbine having a 200-inch runner with direct connected generator would probably be very close to the required economic size. This result was substantiated by later and more accurate work. This entire investigation, therefore, has centred around units of this size.

**Number and Power of Turbines.**—The power produced by the selected 200-inch runner, at full gate, under a head of 10 feet is slightly over 3,000 horse-power. This is approximately the lowest head that is likely to be reached during a period of low tides under any reasonable scheme of operation. At such a low head, best turbine gate coincides with full gate. The efficiency of the generator will be slightly less than its maximum, since it is operating at less than full load. This, however, is unavoidable. The number of units required in the power-house, therefore, is fixed by the maximum amount of power which can be obtained from the lowest tides with machines of the type described. Using the production of firm power, at 100 per cent load-factor, as the criterion for station capacity, as explained in Section 9, this worked out to be 28 units. Two additional units, making a total of 30 in all, have been included in the estimates. These additional units provide spare machines for use when repairs are necessary and when routine overhauling is being carried out.

With increasing heads and output above the 10 feet just discussed, best turbine gate falls back somewhat, until at a head of 34 feet it is at about 80-85 per cent of the full gate opening. During such a manoeuvre, the actual efficiency at best gate has arisen to a maximum and fallen back again, but over quite a narrow range, however, so that a high efficiency is maintained throughout. Plate No. 6 shows this point very clearly.

Under the maximum head which will exist during the highest tides, viz., 43 feet, the 200-inch runner will produce approximately 20,000 horse-power at full gate. The full gate power for the turbine will therefore vary from about 3,000 horse-power to 20,000 horse-power and the corresponding generator outputs would be 2,600 kva and 17,700 kva, both at 80 per cent power-factor. A suitable generator capacity would lie somewhere between these two extremes. For the purpose of this report, a generator capacity of 9,000 kva at 80 per cent power-factor has been selected. This will enable the generator to supply power corresponding to turbine operation at best gate under a net head of approximately 26 feet. In other words, when the station is operated to produce the maximum possible amount of power in each tide at 100 per cent load-factor,

there will be no cut in station output due to insufficient generator capacity, for all tides having a range of about 32 feet or less. This corresponds to 35 per cent of the time as will be seen by reference to the duration curve of Plate No. 5. As the tidal range increases above 32 feet, there will be a slight diminution in station output, due to the limitations imposed by the generator, gradually increasing in amount until the highest tides are reached. This point is amplified in the power studies of Section 9 of this report.

**Description of Turbines.**—The usual single runner, vertical shaft turbine which has been developed for fresh-water hydro-electric stations will be eminently suitable for the proposed tidal power-house. In order to reduce to a minimum the amount of metal exposed to sea water, it is possible that a design for the stay ring might be worked out similar to that proposed for Passamaquoddy. This involved the replacement of the all-metal stay ring with a series of steel stay vanes suitably designed for embedment in the concrete of the scroll case. This arrangement has the further advantage that if any of the vanes become corroded sufficiently to impair their strength, they could be replaced with a minimum of expense. The cost of renewing a stay ring of the conventional design would be very heavy indeed.

A cross-section through the unit is shown on Plate No. 14. The scroll case is of reinforced concrete design, as is also the draft tube. The latter is provided with a steel plate liner at the throat section. Because of the very large quantities of water used by these units, it will be necessary to exert every effort to secure a design for the scroll case which will guide the water to the turbine not only with a minimum of energy loss, but also with the proper distribution and direction of velocity around the periphery of the gate circle. Similarly, the design of the draft tube must be worked out to secure the greatest possible amount of regain of the kinetic energy in the water as it leaves the turbine runner. Both of these requirements can be greatly advanced by carefully conducted tests of scale models of the proposed design.

Periodic and thorough inspection of all submerged metal work will be required. Hence, it will be necessary to include in the design adequate facilities for quickly and completely unwatering the turbine scroll case and draft tube. The full details of such facilities have not been shown on Plate No. 14. Appropriate sums, however, to cover their cost have been included in the estimate.

**Inlet and Outlet Gates.**—The function of these two sets of gates has been described in Section 1 of this report. In order to give an idea of the duty which these gates will be called upon to perform, the following figures have been taken from the power studies, which latter are described in Section 9. With the generating station producing power at 100 per cent load-factor, the maximum quantity of water handled by the gates for various tidal ranges is given hereunder:—

| Tidal Range | Maximum Discharge Through | |
|---|---|---|
| | Inlet Gates | Outlet Gates |
| 25 feet | 392,000 c.f.s. | 335,000 c.f.s. |
| 30 " | 445,000 " | 385,000 " |
| 35 " | 497,000 " | 405,000 " |
| 40 " | 540,000 " | 420,000 " |
| 45 " | 572,000 " | 437,000 " |
| 50 " | 594,000 " | 455,000 " |

As will be seen by reference to Plates Nos. 7 to 10, the diagrams of which represent conditions for a typical high and a typical low tide, the discharge through each set of gates is zero at the instant of opening, quickly

rises to a high value, gradually approaches the maximum and then down to zero again, at which point the gates are closed. The maximum discharges through the gates, as given by the above figures and as shown on the Plates, are very great quantities indeed. Perhaps a better appreciation of what is involved can be given by the following figures:—

| Tidal Range | Total Volume of Water Passing Through | |
|---|---|---|
| | Inlet Gates | Outlet Gates |
| 25 feet | 98 sq. mi. ft. in 2·7 hrs. | 94 sq. mi. ft. in 2·8 hrs. |
| 30 " | 104 " 2·5 " | 106 " 2·9 " |
| 35 " | 108 " 2·4 " | 115 " 2·9 " |
| 40 " | 112 " 2·1 " | 120 " 2·9 " |
| 45 " | 115 " 2·0 " | 121 " 2·8 " |
| 50 " | 118 " 2·0 " | 108 " 2·5 " |

Taking the 35-foot tide as an example, it will be necessary when such a tide occurs for the inlet gates to pass a volume of water equivalent to a depth of 108 feet on one square mile in the short space of 2 hours and 24 minutes, while the outlet gates must pass a volume of water equivalent to a depth of 115 feet on one square mile in the short space of 2 hours and 52 minutes.

The gates and their connecting channels inevitably present some obstruction to these flows, the amount of such obstruction depending on their design.

With no obstruction whatsoever, the level of tide-water would coincide with the water level in the basins during both the filling and emptying operations. This, of course, is impossible since the gate openings and connecting channels will offer resistance to flow with the result that during the filling of the high-level basin, the water level in it will lag behind tide-water level, while during the emptying of the low-level basin the water in it will likewise lag behind the level of the falling tidewater in Shepody Bay. This feature is shown by Plates Nos. 20 to 26, inclusive, and by Plates Nos. 36 to 42, inclusive, which delineate the water levels in the high-level basin, the low-level basin and in Shepody Bay during one complete tidal cycle. Because of this lag, it is impossible either to fill or to empty the basins completely to the level corresponding to the top or bottom of the tidal range, as the case may be, and the degree of inability to do so is measured directly by the resistance to flow offered by the gates and connecting channels. In other words, the resistance offered by the gates and channels causes not only a loss of head during the filling and emptying operations but also a decrease in effective head on the turbines acting over and lasting for each complete tidal cycle and the amount of this permanent loss of head or fall between the upper and lower basins is governed directly by this resistance. Accordingly, the gates and connecting channels must be designed for the lowest possible loss of head compatible with economy of construction.

There is one exception to the general statements given above, and this relates to the action of the outlet gates under tides having ranges of about 40 feet or greater. In order to fully empty the low-level basin during these higher tides, the sills of the outlet gates must be set much lower than the actual figure of —35·0 used in this report. Additional excavation, additional concrete and heavier gates would have been required, and as compensation therefor extra power would have been secured. It would obviously be uneconomical to push the design to the limit in order to extract the last horse-power from the highest tide. Accordingly, an intermediate design has been selected to provide a reasonable balance in this matter.

Vertical sliding gates, capable of being lifted clear of the water passages, are the only type which answer all of the requirements for the inlet and outlet

gates. They can be made sufficiently large and the supporting structure adequately streamlined so that losses are reduced to a reasonable minimum. The conditions outlined above require the use of 27 inlet gates each 50 feet wide and 30 feet high, and 34 outlet gates each 40 feet wide and 20 feet high. The sills of the former have been set at Elevation —20·0 to provide a minimum of resistance at the low tides, while the sills of the latter have been set at Elevation —35·0 as previously explained. The lifting speed for the gates has been set at 2·0 feet per minute in the case of the inlet gates and 1·33 feet per minute for the outlet gates so that either lifting operation will take place in 15 minutes. Furthermore, since in both cases the gates will always be lifted under balanced water conditions, or very close to it, the power required for operation will be quite small.

The gates would be of the usual steel plate and girder or truss type, provided with fixed end rollers and electric motor-driven hoisting mechanism. The gains and gates would be arranged so that electric heaters could be installed at any time should ice formation make them necessary.

The proposed arrangement of the piers and gates is shown on Plates Nos. 15 and 16. As will be seen from these illustrations, each gate has its own hoisting mechanism. Two complete sets of emergency unwatering gates, for both the inlet and outlet gates, have been included in the estimates. These will enable any two inlet gates and any two outlet gates to be completely unwatered either for routine inspection of the bedded parts or for doing such upkeep or making such repairs as may be found necessary from time to time. These emergency gates are handled by the electric travelling gantry shown on the Plates.

## 7. DESIGN AND CHARACTERISTICS OF ELECTRICAL INSTALLATION

**Generators and Switching Arrangement.**—For the generating equipment, we propose the installation of thirty vertical water-wheel generators of the self-ventilated type, each 9,000 kva, 0·8 power-factor, 3 phase, 60 cycle, 13,800 volts. It may be reasonably assumed that six transmission lines will be required to handle the output of this development. It is therefore proposed that power from the generators be similarly sub-divided, and that the units be bussed in groups of five, each group feeding through a transformer bank to a high-voltage switching station from which the lines will radiate. This is shown on Plate No. 11. Under this arrangement, installation of the power-house and switching station equipment would be made in six successive stages, each of which would add 45,000 kva to the station capacity, an amount which is consistent with the normal growth of a development of this magnitude.

**Transformers.**—The conventional type of oil-insulated water-cooled transformer is not suitable for application where sea water is used as a cooling medium, owing to the presence of entrained matter. A type of cooling in which the transformer oil is pumped through a separate heat-exchanging chamber is therefore proposed, the design of the exchanger being such that the tubes will be readily accessible for cleaning. With this system, the transformer oil is pumped through the heat exchanger at a pressure slightly in excess of that used for the cooling water, thereby preventing any possibility of water leaking into the transformers. Each of the six transformers would consist of a 45,000-kva, 3 phase unit, wye-connected on the high-voltage side to furnish a line potential of 132 kv.

**Switching Station.**—From the transformers, power would be carried to the high-voltage switching station over six lines strained from structures on the power-house roof. It is proposed to locate the switching station on the north shore of the intake channel, as this forms a suitable point from which to radiate the outgoing lines. The station would be built in two sections, each comprising

a ring bus to which three transformer banks and three lines would be connected as shown on Plates Nos. 11 and 12. An emergency tie breaker would be provided to permit the interchange of power between the ring busses under exceptional conditions. It is clear that load conditions on the various lines will have a direct bearing on the arrangement of connections, and some departure from the set-up shown on Plate No. 11 may be necessary. However, the important advantage of being able to isolate any part of the equipment for servicing without interfering with the operation of the balance of the system should be incorporated into the layout, so far as conditions permit.

## 8. DESIGN OF PERMANENT WORKS

**General.**—For a two-basin scheme at Hopewell Cape, the upper and lower pools will be formed by the estuaries of the Petitcodiac and Memramcook Rivers. The question arises, therefore, as to which of these should be the high-level and which the low-level basin.

There are several points to be considered in this connection. In the first place, as explained in the section entitled "Navigation", Moncton and Hillsborough on the Petitcodiac River have always been ocean ports; whereas navigation on the Memramcook has been confined, in the past, to a few small fishing vessels and very light craft. With the Petitcodiac estuary as a high-level basin and the Memramcook estuary as a low-level basin, the existing ocean-going navigation on the Petitcodiac can be retained, while the small amount of traffic on the Memramcook can be satisfactorily taken care of by other means. If the estuaries were reversed, as to function, Moncton and Hillsborough would lose permanently their status as ocean ports. For this reason alone, it would be necessary to make the Petitcodiac estuary the high-level basin.

From the viewpoint of the tidal scheme, "per se", there are other arguments reinforcing the above conclusion. The elevation of the turbine runner is determined and fixed by the low-tide levels. On the discharge side of the turbines all the variation in water level, regardless of its amount, will occur above these low levels, and the design of the structures can easily be arranged to accommodate it. On the inlet side of the turbines a quite different condition occurs. Because of its physical size in relation to the normal tidal range, the roof of the turbine scroll has very little submergence. Consequently, if the headpond drawdown is great, as it would be if the ratio of upper basin area to lower basin area were very much less than unity, the designer would be under the necessity of lowering the scroll intake sufficiently to prevent the admisssion of air. This is expensive, particularly if additional excavation were involved, and it also decreases the efficiency of power generation.

At the Petitcodiac site the ratio of the area of the Petitcodiac and Memramcook estuaries is approximately 9·5 to 1. Even with dredging in the Memramcook to the greatest practical extent, this ratio can only be reduced to approximately 2·2 to 1. On this score, therefore, the Petitcodiac is favoured as the high-level basin.

And finally, a brief study of the general layout as shown on Plate No. 13 will demonstrate that it is impossible to re-arrange the structures so as to allow the Memramcook to be used as a high-level basin, except with enormously increased excavation quantities.

**Main Structures.**—As shown on Plate No. 13 the main physical components of the proposed development may be classified as follows:—

(*a*) The power-house with its attendant waterways, including the headwater channel from the Petitcodiac River basin, and its tailrace channel to the Memramcook basin.

(*b*) The main inlet gates which will permit the tidal waters to flow into the Petitcodiac basin from Shepody Bay.

(*c*) The main outlet gates which will permit the spent water from the turbines to flow from the Memramcook basin into Shepody Bay.

(*d*) The rock-fill dams bulkheading the Petitcodiac and Memramcook estuaries.

(*e*) The necessary lock section which will permit ships to proceed as they have done in the past up the Petitcodiac River to Hillsborough and Moncton.

**Power-house.**—Obviously, the location of the power-house is predetermined to the extent that it must be constructed on Fort Folly Point, and on a solid rock foundation. Such a location involves a large amount of rock excavation, both for the power-house proper and for its entrance and tailrace channels, and the cost of excavation is the most important factor in determining the final location of the structure. For this reason it has been located as closely to the existing waterline as is consistent with the important physical features entering into its design, such as the depth and length of the draft tubes, the over-all length of the power-house, and the feasibility of constructing cofferdams to such extent as will allow all work in connection with the power-house excavating and concreting to be done in the dry.

The power-house structure will house 30 generators, each rated 9,000 kva, at 0·8 power-factor, spaced 62 feet centre-to-centre. The main operating floor will be at Elevation +14·0, the centreline of turbines at Elevation —9·0, and the draft tube floor at Elevation —22·0. The headworks, which in this instance form an integral portion of the power-house structure, will be equipped with electrically-operated head gates, the necessary racks and emergency gates, all with the necessary checkliners, hoists and other essential facilities.

The superstructure will be of steel frame construction, with concrete or brick curtain walls to be decided upon later. Sufficient space will be provided for all necessary control apparatus.

The highway, crossing the Petitcodiac dam, will continue along the headworks section of the power-house, and along the west bank of the Memramcook estuary, paralleling at least for some distance the construction railway.

The main elements of the power-house structure as above described are indicated on Plate No. 14.

**Inlet and Outlet Gates.**—If foundation conditions had been favourable in the beds of the Petitcodiac and Memramcook estuaries, then the natural location or site of the inlet gates to the Petitcodiac basin would have been in the centre of the Petitcodiac dam structure. Likewise, the natural location for the outlet gates from the Memramcook basin would have been in the centre of the Memramcook dam structure. With the type of rock-fill dams to be described in the following sub-sections, such locations are obviously impracticable, both from the construction and operating standpoints. The general theory dealing with the construction of the rock-fill dams at the mouths of the two estuaries admits continual sinkage of some considerable degree during the construction period, and also quite possibly some further degree of sinkage for the first several years of operation. With the gate structures located in the river areas proper, this later sinkage would most certainly have the effect of throwing the gate sills out of direction, and the gate liners out of plumb, to such extent that it would not be possible even to lower or raise the gates within a short time after their installation.

Logically, then, it became necessary to locate the inlet gates to the Petitcodiac basin on the solid rock ledge extending from Fort Folly Point west to and below the waterline of the Petitcodiac. Likewise, it became neces-

sary to locate the outlet gates from the Memramcook basin on solid ledge rock encountered on the east bank of the Memramcook, both as illustrated on Plate No. 13.

The inlet gate structure located as shown on the general plan arrangement, Plate No. 13, consists of 27 sluice gates, each 50 feet wide by 30 feet high. Intermediate piers of reinforced concrete measure 15 feet in width by 120 feet over-all in length. A mass concrete abutment, with the necessary wing walls, ties one end of the gate structure to the Petitcodiac rock-fill dam, and the other end ties into the power-house structure. The highway continues from the top of the rock-fill dam, across the inlet gate structure and thence along the upstream side of the power-house. A part plan and typical cross-section of the inlet gate structure are shown on Plate No. 15.

The outlet gate structure, also located as shown on the general plan arrangement (Plate No. 13) consists of 34 sluice gates, each 40 feet wide by 20 feet high. Intermediate piers of reinforced concrete measure 12 feet in width by 133 feet in length. A mass concrete abutment, with the necessary wing walls, ties one end of this structure to the Memramcook rock-fill dam, while the other end extends into and seals off the excavated area along the east bank of the Memramcook estuary. A part plan and typical cross-section of the outlet gate structure are shown on Plate No. 16.

All work, except certain incidental rock excavation, in connection with the construction of the inlet and outlet gate structures must be completed in the "dry", and the construction of temporary cofferdams for unwatering purposes is therefore a necessary and expensive item in the cost of these two main elements of the project. Wherever feasible, timber cofferdams will be constructed on ledge rock at existing rock elevations, and the rock foundation under the cofferdams must be drilled and blasted, partly in the "wet", after the temporary cofferdams have been removed. At the site of the outlet gates, timber cribs are feasible only for a minor proportion of the total length of the cofferdam structure, and the steel-sheet-piling cellular type cofferdams must be constructed for the considerable remaining distance.

**Lock.**—The same argument in respect to suitable foundations applies to the lock, and it is therefore located at the west end of the Petitcodiac dam adjacent to the existing Government Wharf at Hopewell Cape.

The general layout of the lock section is shown on the general plan arrangement, Plate No. 13. As indicated, the lock proper will measure 60 feet in width and approximately 400 feet in length, between gates. The concrete lock wall section on the river side will measure approximately 1,200 feet in length, while the dock area on the land side will measure some 2,200 feet in length. This will provide sufficient length for any vessel proceeding up or down the Petitcodiac River to tie up at either the downstream or upstream side of the lock, before actually entering and passing through the lock.

The construction of the dock area involves a concrete gravity wall, with the space between the wall and the shoreline filled in with broken stone or gravel, up to Elevation +36·0.

The construction of the lock wall, the lock floor and the corresponding length of dock walls, must be done in the "dry", and the construction program therefore will include provision for cofferdamming. The remainder of the dock walls, both upstream and downstream, may be constructed without the use of cofferdams. In these sections the length of wall to be poured as a unit would be determined by the concrete placing equipment available. The form sections would be pre-fabricated in such a manner that they could be erected in a minimum of time, and the concreting would proceed at such a rate that the surface of the concrete in the forms could keep ahead of the incoming tide.

Accessory structures and equipment associated with the lock and dock construction would include the necessary gates, dock and lock lighting, haulage units and channel markers. A lift or bascule type bridge would be constructed, which would carry the highway over the lock on to the top of the rock-fill dam at Elevation +36·0.

**Rock-fill Dams.**—A study of the rock contour plan at the projected dam sites will at once make evident the fact that ordinary design and construction procedure cannot be used in the construction of these bulkheads, which will seal off the outlets of both the Petitcodiac and Memramcook estuaries. It is obviously impossible to cofferdam, excavate and construct the conventional concrete bulkhead at either of these locations.

As indicated by the borings made by the Federal Department of Public Works, a very considerable depth of clay, sand, and gravel, all in layers of variable thickness, overlies the ledge rock strata below.

The construction of the power-house at Fort Folly Point, with its attendant excavation, and the excavation of the inlet channel upstream from the powerhouse, will provide rock to an amount in excess of that required for the construction of the two rock-filled dams. Some considerable portion of this excavated rock will have to be stockpiled and rehandled. Also, from the construction and time standpoint, it will quite probably be desirable and necessary to open a quarry on the Hopewell Cape side of the river. There is no question, therefore, of the availability of sufficient rock for the construction of rock-filled barriers of adequate cross-section.

The base of these dams would initially rest on the clays, sands, and gravels, which form the material overlying the rock stratum, and therefore the matter of settlement will determine the final basis of design, rather than the usual factors of overturning and sliding.

The following two general theories, originally explored during the investigation of the Passamaquoddy Tidal Power Project, and dealing with the settlement of rock-fill structures, are applicable to any study dealing with settlement in the overburden comprising the immediate foundation beneath the proposed Petitcodiac and Memramcook rock-fill dams.

First: The total weight of the rock-filling of the dam structure may displace the clay layers in the underlying foundation material, causing them to flow laterally under the overlying structure and rise in more-or-less regular waves on either side of the structure, and outside the limits of the structure base. This movement of the clay stratum is dependent entirely upon the consistency of the clay, the superimposed weight of the structure above, and the ratio of width of structure base to the depth of the individual clay layers. Theoretically, the base of the dam could be spread over such width that the clay waves might be suppressed, and thus limit the amount of settlement under the structure, but such a procedure would seem both impractical and more expensive than to allow the rock-fill structure to settle continuously until a solid base is reached, either on the rock strata beneath, or at some intermediate point, depending on the nature of the material overlying or immediately adjacent to the rock strata.

Briefly, the method of construction under this latter alternative would be the gradual addition of rock fill to compensate for this complete or nearly complete settlement.

Second: The individual stones in the fill, under their own weight and the superimposed weight of the stones above them, may be forced into the clay, and the latter would work up between the stones above, until a great deal of the clay under the dam structure is permeated with stone fragments, while at the same time the displaced clay has been forced up into the voids of the rock fill to a level possibly above that of the adjacent river bottom.

From the practical standpoint it would seem obvious that either one or both of these two types of settlement would be encountered during the construction of a rock-fill dam, in either the Memramcook or Petitcodiac estuary bottoms.

The final results of various tests carried out by the Soil Laboratories set up in connection with the Passamaquoddy Project, prove that the base of the dam will settle roughly into a double-reversed peak, leaving a wedge of undisturbed clay under the centre of the dam structure above, while the inner and outer toes of the rock-fill structure will settle progressively until such time as the portion submerged in the clay will approach or equal in height the portion of the rock-fill structure above the bottom of the channel.

In other words, if the dam is constructed approximately in a triangular shape, the resulting settlement will probably approach an inverted parabolic curve. This type of settlement is shown graphically on Plate No. 17, which indicates the typical cross-sections proposed for the Petitcodiac and Memramcook rock-fill dams.

It is a fairly well-established rule that broken stone dropped in a pile, either in air or in water, will assume a slope of about 45 degrees, or about one vertical upon one horizontal. It is evident, therefore, that a slope of this degree is a limiting slope, and that at any steeper slopes rock will tend to become dislodged and roll to the bottom.

There are various factors which will have a tendency to flatten the side slopes of the rock-fill structure, whether or not the side slopes have been designed at or about 45 degrees, the normal angle of repose. The final slopes will first be dependent upon the amount and direction of settlement. The laboratory studies, mentioned previously, have indicated that the bottom of the side slopes would be pushed outward and upward by the process of settlement. The slope will also be controlled to some extent by the method of placing the stone.

When it is possible to deposit stone carefully, piece by piece, upon stone already in place, a slope approximating the natural angle of repose could result. When placing must be done by dumping from scows in comparatively large quantities, some considerable portion of the stone so dumped will slide down the incline slope, thus tending to flatten it and to widen the base.

Actual placing operations at the sites under consideration must also continue throughout the tide cycle, as it obviously would be impossible to place stone only during the lower reaches of the tide. Therefore the resistance of the stone to displacement under water is less than would be the case if the stone were piled up in the open.

When the water level lowers, the stone above the lowering level is therefore relieved of the buoyant effect of the water, and the result is to place a greater weight on the submerged stone below. This also tends to result in settlement, some of the submerged stone doubtless being displaced, rolling down the slope and further flattening it.

In all probability the velocity of the current will have some considerable effect on the final slope of the fill. Experiment has shown that under velocities approximating 5 feet per second, fine material will begin to wash out of a gravel bank. If the velocity becomes greater than 5 feet per second, increasing amounts of gravel will start to move, and at velocities approaching $5\frac{1}{4}$ feet per second, a gravel bank may become unstable. The significance of such experiments then would indicate that the velocity of the tidal currents will have some appreciable effect on the slopes of the structure, and will govern to some extent the minimum size of stone that may be placed on the outside of either the upstream or downstream slopes.

The actual side slopes that will obtain in the two dams in question are not particularly important, as there is an excess of rock excavation available at the cost of rehandling only, which may be used to widen the base, to flatten the slopes, or to do both. For the purpose of determining the minimum amount of stone which must be dumped into these two dam sections, the top width of the dam has been fixed at 100 feet, and the side slopes have been fixed at 1·75 horizontal to 1 vertical.

The main problems associated with the construction of these rock-fill dams in the Petitcodiac and Memramcook estuaries are therefore as follows, in the order of their importance,—

(1) The construction of an impermeable core.

(2) The possibility of obtaining, from the excavation material, large blocks to be used in the outer portions of the rock fill.

(3) The problems associated with the placement of rock fill in either one or other of the two dams.

It is evident that such a rock-fill structure must be constructed in horizontal layers, the exact depth of which may be only determined by experience gained in the earlier stages of the development.

Purely from the construction standpoint, it would obviously be simpler to build the rock fill to its full height at the river banks, and thus continue in vertical layers across the width of the river. By so doing, however, inevitably the open portion of the river through which the tide will flow in and out, becomes progressively smaller, and tide velocities become progressively greater, until a stage is reached when these velocities equal or exceed the scouring point in the river bottom.

As noted previously, Plate No. 17 indicates typical sections through the rock-fill dams in the Petitcodiac and Memramcook dams, respectively, with crest elevation at +36·0 (Geodetic Datum).

The various operations involved in the construction of such sections are indicated on the Plate above referred to and may be enumerated as follows:—

(1) The construction in horizontal layers of the bottom layers of the dam; the sections composed of quarry-run rock being placed first, the fine core material being placed second, and the large heavy stone facing both upstream and downstream being placed third. This operation would continue until the surface of the fill had reached mean sea level. The quarry-run rock and outside facing could then be built up until its crest reached approximate Elevation +16·0 (Geodetic Datum) but the central core of fine material would not be added until Operation 2, as detailed in the following paragraph, would be completed.

(2) Operation 2 would entail the driving of 80-foot-long (maximum) steel sheet piles of suitable section from Elevation 0·0 to Elevation —80·0 in the Petitcodiac dam, and 60-foot-long (maximum) steel sheet piles from Elevation 0·0 to Elevation —60·0 in the Memramcook dam.

As shown on Plate No. 17 the bottom of the 80-foot piles, in the Petitcodiac dam, would reach the top of the clays, sands, and gravels as compacted and as shaped by the sinkage of the quarry-run rock, and that the bottom of the 60-foot piles, in the Memramcook dam, would penetrate for some few feet this compacted layer.

It would appear that with the quarry-run rock built up to Elevation +16·0 the sinkage or submergence of the fill in the soft material above the rock stratum will have been completed to a major extent.

(3) Operation 3 entails the completion of the quarry-run rock, and the completion of the heavy stone outer faces to crest Elevation +36·0, and the completion of the centre core of finer material to Elevation +26·0.

(4) Operation 4 entails the driving of two rows of lighter steel sheet piling from Elevation +26·0 to approximate Elevation —5·0; these piles therefore being 31 feet long.

(5) In Operation 5, the space between the two rows of steel sheet piles driven under the preceding operation would then be excavated down to Elevation —5·0, timber bracing being used to prevent the opening from being closed up by the pressure of the rock on either side.

(6) Under Operation 6, two alternatives present themselves for the completion of the central core.

The first alternative would be to completely fill the space between the two rows of steel sheet piling with concrete, extending the concrete up to the final crest elevation at +36·0.

The second alternative would be to splice the 80-foot piles below with shorter piles, which would be extended to Elevation +36·0; fill the intervening space between the piles forming the trench, with fine material, and then salvage the two rows of 31-foot piles, which could be used over again in the Memramcook dam.

The cost of the dams as set up herein includes the cost of filling the excavated trench with concrete. It is considered that the second alternative, i.e., splicing the 80-foot piles and finally salvaging the 31-foot piles, while somewhat cheaper in cost, would not be as efficient in providing a waterproof membrane, which could later be extended vertically if proven necessary, due to abnormal sinkage.

(7) Operation 7 would entail the completion of the fine core material in the central part of the dam; the trimming of all slopes above the low-water tidal elevation; the replacement of any portion of the structure which has progressively sunk during the last three operations, and the road surfacing, fencing, etc., which will be necessary at the crest elevation.

It may be seen from the above description, and from the details shown on Plate No. 17, that the uncertainty in connection with the quantities involved in such a dam structure is the sinkage of quarry-run rock on either side of the central section of the dam. The sinkage of the fine material in the core section, while of some considerable amount, will probably not be in the same proportion as the sinkage of the quarry-run rock.

The structures as now described and shown on Plate No. 17 therefore theoretically include a waterproof layer of compacted clay and sand, extending from the rock floor to the bottom of the central steel sheet piling; a waterproof membrane of steel sheet piling extending above the compacted clay, to mean sea level; a waterproof section of solid concrete formed by two rows of steel sheet piling extending from Elevation —5·0 to Elevation +36·0 with the tops of the latter-mentioned piles, however, at Elevation +26·0, or as mentioned under alternative two of Operation 6, in place of the waterproof section of solid concrete, an upper section of steel sheet piling spliced to the longer piles below and carried up to Elevation +36·0.

Three possible methods of placing the rock fill may be listed as follows:—

(*a*) Dumping from scows.

(*b*) Placing by cableway.

(*c*) Placing by heavy duty trucks.

In connection with the first alternative, the main obstacle will be the rapid tidal currents experienced during certain periods of the tidal range, and the difficulty of properly anchoring and controlling the position of the scow at the time of dumping.

The main difficulty in connection with the second alternative is the comparatively long span which obtains across either the Petitcodiac or Memramcook estuaries, and therefore with any cableway installation it will be necessary to construct an intermediate pier or crib in the centre of the river, from which an intermediate tower may be erected.

The above points are mentioned to illustrate that whatever scheme of construction may ultimately be devised, there are certain obstacles and hazards that must be overcome, and which must be taken care of in the cost estimates dealing with such an installation.

Obviously, as the rock-fill section is built up to a point which is above the low tidal range, heavy duty trucks may be used, and the problem of placing the rock then becomes somewhat easier and less expensive.

## 9. AVAILABLE POWER

**Power from the Tides.**—Consideration is given in this section to two methods of operating the proposed tidal power plant. Under the first of these methods the maximum amount of constant continuous power is obtained from each tide. In other words, the station output, in horse-power, is constant during each tidal cycle, but changes from tide to tide, as the range varies, in order that the maximum amount may be generated from any particular tide. Under the second method, the units are operated to produce the maximum possible amount of energy, in kilowatt-hours, from each tide. In this case, the horse-power output varies continuously over the tidal cycle, in order that the required maximum energy may be produced.

Subject to the limitations of the desired result, each of these two methods of operation makes a maximum use of the power potential of each tide.

There are, of course, an infinite number and variety of operating methods intermediate between the two extremes just described. Each could be so devised as to make a maximum use of the power potential of any tide, or some lesser use as may be desired. It is believed that no useful purpose would be accomplished by working out any one or more of them since they do not lend themselves to any orderly classification and the selection, of necessity, would be purely fortuitous. The results of a study of a few isolated operating methods would accordingly be meaningless.

Moreover, no attempt has been made to match generated power with any particular pattern of load demand. The reason for doing so is given further on in this section and will appear more evident after a perusal of the immediately-following discussion.

**Maximum Constant Continuous Power.**—The series of power studies to determine the maximum amount of firm power at 100 per cent load-factor which can be extracted from each tide is described briefly hereunder and is followed by a commentary upon the results.

The first item of information necessary for any power study is a set of tidal curves, for typical conditions, such as those illustrated on Plate No. 3. The mathematical representation of such curves is an extremely complex function, since tidal movements are the summation of a number of factors, none of which can be expressed by a simple mathematical relation. Lacking actual tidal curves, therefore, it is not practical to derive a set of tidal curves comprising, for example, the maximum and minimum tides of the duration curve of Plate No. 5 and several intermediate tides having whole number ranges and

selected at more-or-less regular intervals between these two extremes. Recourse was therefore made to the charts of 1919 and from these were selected curves of tidal ranges of 25·0, 28·6, 33·1, 37·52, and 41·95 feet, as representing the widest selection possible with appropriate intermediate values. This selection, however, came far short of covering all the ranges included in the full 18-year cycle. In order to extend these selected tides, therefore, it was assumed that the tidal chart for the 25·0-foot tide could be used with a doubled ordinate scale to represent a 50·0-foot tidal range and that similarly the ordinates for the 33·1-foot tide could be multiplied by two-thirds to produce a 22·1-foot tide.

Additional pre-requisite items of information are the operating characteristics of the turbines over the full range of heads and gate openings, the hydraulic characteristics of the entrance channel with the inlet gates closed and water flowing from the high-level basin directly to the turbines and also with the inlet gates open and water flowing from Shepody Bay through the inlet gates and from thence part going to the high-level basin and part direct to the turbines. The hydraulic losses through the inlet gates themselves must also be established. Similarly, the hydraulic characteristics of the combination of the outlet channel and outlet gates must also be calculated.

The amount of the rise and fall in each of the two basins is dependent on their respective water areas. These latter, in turn, vary with the elevation of the water surface and one of the principal reasons for making the surveys described in Section No. 4 was the securing of this information. Since it is a matter of general interest, the relation between basin area and elevation is given graphically on Plates Nos. 18 and 19 for the Petitcodiac and Memramcook Rivers, respectively. The former of these shows present areas since no dredging for the purpose of obtaining additional basin area is contemplated for that river. Plate No. 19 shows the water surface areas of the Memramcook River as they would be after the maximum amount of dredging had been performed. These curves indicate that even under this condition the ratio of the area of the Petitcodiac basin to the Memramcook basin varies from about 2½ to 1 to about 2 to 1 depending on the respective elevations in the two basins, as they occur, during the processes of filling and emptying. Had the Memramcook basin areas been larger and this ratio reduced to 1 to 1, a considerable increase in power from the site could have been obtained.

In making the power studies, the actual tidal curves were used without attempting to modify them so that the extreme low water at the beginning and end of each tidal period would come artificially to the same elevation. This arrangement introduced an additional complication into the mathematical work and explains why the curves for head and discharge do not always have the same value at the end of the cycle as at the beginning. The effect of the normal diurnal variation in tidal height was therefore not excluded from the studies.

Because of the rapidly changing levels in the high and low level basins and in Shepody Bay and of the necessity for calculating very carefully the effects on these levels of the enormous discharges through the control gates, it was necessary to employ a step-by-step method of computation. As will be seen by reference to Plates Nos. 20 to 26, there are three separate and distinct periods of operation for any one tidal cycle. During one of these periods, both sets of control gates are closed, in another the inlet gates only are open, and in the third the outlet gates only are open. It was found that any attempt to apply a method of average values to each of these periods only led to extremely erroneous results. Consequently, the 12 hours and 25 minutes of the tidal cycle were divided into an appropriate number of small intervals of time, and the changes in water level, discharge, etc., calculated for each of these small intervals.

The calculations for each interval are performed by setting up the equations governing the relation between quantity of water required to generate the set amount of power, the corresponding drawdown in the high-level basin, the corresponding rise in the low-level basin and the consequent net turbine head. To these equations must be added the condition for flow through the control gates when these latter are open. The number of dependent variables involved is greater than the number of equations, consequently no direct solution is possible and the correct answer can only be obtained by repetitive trial and error until a true result occurs.

The results of the above study are given in three sets of Plates. Plates Nos. 20 to 26 show for one tidal cycle the variations in the elevation of the water in the high-level basin, the low-level basin and in Shepody Bay for each of the seven chosen tidal ranges. They also show the time of opening and of closing the two sets of control gates. In these studies all of the gates are assumed to be instantly opened and instantly closed, although this would not be physically possible. This assumption has no significant effect on the results of the study and was made to bring the total amount of time involved in the computations to within practical limits.

Starting at low water, these Plates show graphically the drop in level in the Petitcodiac basin and the rise in the Memramcook basin as water is passed through the turbines in the generation of power. When the level in Shepody Bay coincides with that of the high-level basin, the inlet gates are opened. Shepody Bay now rises above the high-level basin by the amount required to force water through the inlet gates and the entrance channel, and the vertical difference between the two graphs represents these energy losses expressed in feet of water. Towards the close of the filling period, these two levels tend to coincide and eventually do so after maximum high water has occurred and the tide has started to ebb. In other words, the high-level basin can never be filled to the top of the tide, but always to some lower level, the difference varying from 0·3 feet in the case of the 22·1-foot tide to 0·6 feet for the 50-foot tide. As will be seen by reference to the Plates, this difference represents a permanent and unavoidable loss of head.

All this time the low-level basin has continued to rise as the turbines discharge water into it. When the level in Shepody Bay, as the tide ebbs, coincides with that in the low-level basin, the outlet gates are opened and the Memramcook basin, including the turbine discharge, empties through them into Shepody Bay. Here again, the low-level basin lags behind tidal level by an amount sufficient to force the combined discharge through the outlet channel and outlet gates. It was possible to arrange the outlet channel so that the losses through it are much less than the losses through the entrance channel. Hence, the vertical distance between the two graphs is smaller than the corresponding difference when the inlet gates are open. Also, the low-level basin cannot empty completely before the tide has turned and is on the make again and another permanent and unavoidable loss of head is introduced into the entire operating cycle.

The difference in elevation between the high and low-level basins constitutes the gross head on the turbines. The net turbine head is the gross head less the losses through the power-house water passages. Values of net turbine head obtained in the studies are plotted in the upper left-hand diagram of Plates Nos. 27 to 33. These curves show that the head is a maximum at low water, falls to a low value just before the inlet gates are open, recovers somewhat as the high-level basin is filled and then falls again to a minimum when the low-level basin reaches its peak and just before the outlet gates are opened. Thereafter, the head rises again to its maximum value at low water. The times at which the gates are operated have been placed on Plate No. 27 to illustrate better these features.

Since this study is concerned only with the production of constant, continuous power, the turbine discharge will vary inversely as the head. The lower left-hand graphs of Plates Nos. 27 to 33 show this feature.

The number of units in operation will vary inversely as the head and directly as the discharge in order to produce power at 100 per cent load-factor. The right-hand graphs of Plates Nos. 27 to 33 show how the number of units varies over the tidal cycle. It indicates that for the lower tidal ranges at least, units would constantly be coming either on or off the line at intervals ranging from about 10 minutes to 1 hour. In the studies, when units are taken out of service, it has been assumed that the turbine gates would be completely shut to avoid the great wastage of water that would be required to run them at speed-no-load.

As the tidal range increases, higher heads result and it will be seen from the graphs that more units must be operated to produce the required fixed plant output. This seemingly illogical procedure is due to the fact that the turbine output, which otherwise would have been ample, becomes limited under these conditions by the generator capacity, and consequently more units are required. This condition first becomes evident on Plate No. 29 which gives the conditions for a tidal range of 33·1 feet.

A commentary on the remaining Plates of this series, viz., Nos. 34 and 35 follows hereunder. As a basis for discussing Plate No. 34 it may be said that a number of preliminary power studies were made before the final computations were started. Their primary object was to determine the conditions under which the turbines and generators would be required to perform and, particularly, to fix the range of turbine heads so that a suitable selection could be made from available runner designs. On the basis of this work, a runner was chosen having the efficiency-head characteristics at best gate shown on Plate No. 6. The two curves of Plate No. 34 show the maximum head and the minimum head which occur during the tidal cycle for the entire series of tidal ranges. In other words, the upper curve is the locus of all the points representing the head at low water, i.e., at the slack of the ebb when the outlet gates are closed, and the lower curve is the locus of all the points representing the head at the instant of opening of the outlet gates, these points being taken directly from the upper left-hand graphs of Plates Nos. 27 to 33.

The curves of Plate No. 34 show that the range of heads for the lowest tide varies from a minimum of 9·0 feet to a maximum of 16·5 feet, while for the highest tide the corresponding variation is from 29·2 feet to 43·0 feet. By referring back to Plate No. 6 it will be seen that the turbine has an efficiency at best gate of 90·0 per cent or over for all heads from 10·4 feet to 27·0 feet. According to Plate No. 34, therefore, this range of high efficiency will include the entire variation in heads for all tides from the very lowest up to and including those having a range of 33·0 feet, excepting for a very short period of time during the extreme low tides when the efficiency falls slightly below 90·0 per cent. In other words, during 58·5 per cent of all tides, which period includes all the low tides, when there is a minimum power potential, the turbines generate power, not merely at extremely high efficiencies, but at or close to the maximum possible.

The two curves of Plate No. 35, the final one of this series, show in summary form the results of this study in terms of the output, in horse-power, from the turbines. The left-hand graph shows the power output over one complete tidal cycle, with time as abscissae, for the seven tidal ranges of the study. These curves take the form of straight lines since continuous power is being produced at 100 per cent load-factor.

The right-hand graph of this Plate is a duration curve of power output. From it may be taken the following interesting figures:—

| Per Cent of Time | Turbine Horsepower Equalled or Excelled |
|---|---|
| 100 | 76,000 |
| 95 | 105,000 |
| 90 | 118,000 |
| 50 | 175,000 |
| 10 | 255,000 |
| 0 | 280,500 |

The minimum amount of firm power, at 100 per cent load-factor, is 76,000 turbine horse-power which is the maximum amount of power at 100 per cent load-factor that the lowest tide of the 18-year cycle can generate.

As previously explained, the capacity of the generators begins to limit the station output at a tidal range of 32·0 feet. With higher tides, this effect becomes more pronounced until finally at the very highest tides all the units operate at the point of maximum generator capacity throughout the tidal cycle. This accounts for the horizontal portion of the curve from 0 to $2\frac{1}{2}$ per cent of the time. In other words, an output of 280,500 turbine horse-power is the maximum possible capacity of the station as fixed in this report.

**Maximum Energy Production.**—In the previous study the horse-power developed during any tide was maintained constant at the maximum possible amount. On the other hand, in this study the station is operated with a constant number of units, all working at maximum efficiency, and the output is allowed to vary. The required number of units can be determined by using a mathematical analysis based upon an idealized form of the tidal curve. By this means it was found that 25 units are required for the 22·1-foot tide study, 27 units for the 25·0-foot tide study, 28 units for the 28·6-foot tide study, and more than 28 units for the higher tides. As previously explained, 28 units of the selected size are required to develop the maximum amount of constant continuous power from the lowest tides. This number of units produces a power-house having physical dimensions fitting well with the topographical conformation of the site. It was on this basis that 28 units, plus 2 spares, were chosen as an appropriate number for the purposes of this report. Since more than 28 units are required for maximum energy production from the higher tidal ranges, a limiting factor is introduced thereby.

This study of maximum energy production was carried out with the same fundamental data and using similar methods of calculation as employed in the previous study. The results are assembled in three sets of Plates as before.

Plates Nos. 36 to 42 trace the rise and fall of the water in the two basins and in Shepody Bay. Plates Nos. 43 to 49 show the variations in net head and power-house discharge as in the corresponding series of the previous study. Instead of the graph of number of units operating, however, the present series of Plates indicates the change in power-house output during the tidal cycle. The times of operation of the control gates have been placed on Plate No. 43.

The fact that the shape of the curves of net turbine head is essentially the same for the two studies has no special significance other than indicating efficient use of available basin volume. The curves of total turbine discharge are exactly opposite in shape for the lower tidal ranges, and would be for the higher ranges were it not for the limiting effect of maximum generator capacity. They indicate that power-house operation is diametrically opposed in the two studies. In the previous study, head and discharge vary inversely

to maintain constant power output. In the present study, the reverse relation holds true since the number of units operating is kept constant. Because the study conditions impose no limitations on the amount of power generated, the resulting net effect is a greater drawdown in the high-level basin than happened in the case of maximum constant continuous power. While the average net turbine head is lower in the present case than in the former, the greater quantity of water used more than overcomes this adverse feature, and an actual increase in energy production occurs. Taking the 28·6-foot tidal range as an example, operation for maximum constant continuous power produces a total of 1,153,000 kilowatt-hours for the tidal period. The corresponding figure for maximum energy production is 1,412,000 kilowatt-hours, or an increase of 22·4 per cent.

Plate No. 50 gives the loci of the minimum and maximum net turbine heads. Comparing the curves of this Plate with those of Plate No. 34, it is seen that the maximum heads are slightly less while the minimum heads are decidedly smaller, facts which are explained by the foregoing discussion.

The two curves of the left-hand graph of Plate No. 51 are the loci of the minimum and maximum total turbine output. They are derived from the graphs of Plates Nos. 43 to 49. These curves indicate very clearly the enormous range in output which must occur during each tidal cycle for the method of operation under consideration.

The right-hand graph of Plate No. 51 is a duration curve of energy production for this study. It will be noted that production from the minimum tide is 770,000 kilowatt-hours per tidal cycle; from the average tide, 1,880,000 kilowatt-hours per tidal cycle; while from the maximum tide it is 2,500,000 kilowatt-hours per tidal cycle of approximately 12 hours and 25 minutes. The shape of this curve, as compared with the duration curves of Plates Nos. 5 and 35, is a direct reflection of the imposed limit on station capacity, as previously explained.

**Characteristics of Tidal Power.**—Plate No. 52 is a comparison sheet and shows the amount and variation in power production for the two methods of operation, had the tidal power scheme been built and working during the week of August 1 to 7, 1919. The lower curve is a reproduction, in seriated form, of the tides shown on the chart of Plate No. 3. The upper graphs were not derived by direct calculation but by interpolation from the results of the entire series of computations. They are therefore somewhat approximate but are sufficiently close to the truth to illustrate the immediately following discussion.

The week of August 1 to 7, 1919, embraced a period when the moon was at perigee. Hence, there is a greater variation in the tidal range during this period than there would normally be for a week in which apogee occurred, or for the intermediate weeks. However, such a variation in range will usually take place about once a month, so that it would be a matter of ordinary occurrence in the lifetime of the proposed tidal power plant and must be reckoned with accordingly.

The plate shows (*a*) the diurnal and day-to-day variation in plant output when operated to produce maximum constant continuous power, and (*b*) the diurnal and day-to-day variation in energy production and the manner in which the rate of energy production is subject to continuous change when the plant is operated to generate the maximum number of kilowatt-hours from each tide. Maximum constant continuous horse-power varies from a minimum of 111,000 horse-power to a maximum of 188,000 horse-power, while maximum energy production ranges from a minimum of 1,170,000 kilowatt-hours per tidal cycle to a maximum of 2,000,000 kilowatt-hours per tidal cycle and the concomitant variation in the rate of energy production is from a minimum of 70,000 horse-power to a maximum of 280,500 horse-power.

The Plate also shows very clearly the effect of the approximately 50-minute daily regression in the tide. The cycle of power and of energy production falls back twice every day and so the time relation of this cycle to the 24-hour daily cycle is constantly changing. There can, therefore, never be any effective correlation between power or energy production as discussed herein and any power load depending for its cyclical variation on the ordinary habits of mankind.

It follows that the most effective use to which tidal power could be put would be its inclusion in a large network as a source of energy, provided the difference between instantaneous network demand and tidal power supply could be continuously produced from another source or sources. A further fact also emerges from the previous discusssion; namely, that peak tidal power production in general should not exceed the minimum network demand for maximum economy of operation. In other words, tidal power should represent a small portion of the total system load. If full use is to be made of the power potential of every tide, both the amount of power and the time at which the power may be generated are matters beyond the control of the operator. They are determined by the relative movements of the sun, moon, and earth. In a broad sense, therefore, tidal power is much like run-of-river power, the difference being that the latter is subject chiefly to seasonal changes of comparatively long duration and slow variation, while the former varies rapidly from minute to minute and from day to day. The chief point of similarity is that in both cases power must be generated as and when the potential exists; otherwise, it is lost forever.

## 10. CONSTRUCTION PROCEDURE AND SEQUENCE OF OPERATIONS

Certain main divisions falling within the above category may be listed and described as follows in their proper sequence from the construction standpoint.

**Construction Power.**—The amount of construction power necessary for the orderly prosecution of the work involved depends to some extent on the type of equipment furnished by the general contractor or contractors. Considering the large amount of rock to be excavated, central compressor plants, electrically operated, would be most necessary. Electrically operated pumps would necessarily be installed at strategic locations. It is herein considered that the minimum requirements for construction power would approach 5,000 kilowatts, which would include power for lighting, camp cooking, and other miscellaneous uses. Power of this order would logically be provided by the New Brunswick Electric Power Commission, providing such an amount should be available, and would involve the construction of a transmission line from Moncton to the site. Alternatively, diesel or steam generated power must necessarily be installed at the site.

**Construction Camps.**—Fortunately, suitable locations for the construction camps and attendant services are available on either side of the Petitcodiac River. Initially, camps would be constructed near Fort Folly Point, but as the work progressed, in all probability secondary camps would be provided near Hopewell Cape. Camps would be constructed for a maximum personnel of 5,000. Camp locations would primarily depend on three main factors:—

1st.—Accessibility to the work.

2nd.—Accessibility to fresh water supply.

3rd.—Fire hazard and protection from same.

**Construction Roads.**—Initial road construction would include the extension of the so-called "Dover Road", along the east bank of the Petitcodiac River from Beaumont to Fort Folly Point; the extension of the road along the west

shore of the Memramcook River from Rockland to Fort Folly Point, and the reconstruction of the road from Hopewell Cape leading to the area adjacent to the Government Wharf.

**Construction Railway, Yards, etc.**—The subgrading for the construction railway from Gayton to Fort Folly Point would be started immediately upon award of general construction contracts and the necessary grading proceeded with in connection with the construction of sidings and storage yards adjacent to the power-house site.

**Rock Excavation in Power-house and Head Channel Area.**—Excavation above high-water levels would be started immediately upon award of contract and assembly of plant in this area. Disposal from the area would include initial placement of rock in the bottom of the two dam sections, stockpiling of suitable rock for later placement in dams, and final disposal in selected dumping areas of all rock unsuitable or unnecessary for the rock-fill dams.

**Borrow Pits for Fine Core Material—Rock-Fill Dams.**—Coincident with the initial placement of rock in the bottom of the two dam sections would be location, purchase, and opening of the necessary borrow pits for the supply of fine core material for the rock-fill dams. The logical location of these borrow pits would be on the Hopewell Cape side of the Petitcodiac River and/or the Coles Head side of the Memramcook River. Thus interference with the rock excavation program at Fort Folly Point would be a minimum.

**Cofferdamming and Unwatering.**—Unusually large quantities of timber would be necessary for cofferdam structures. The size of the timber cofferdams makes the use of western fir imperative, and this material could be shipped direct by boat from British Columbia to Hillsborough and there unloaded on barges and taken down the river to the site under suitable tide conditions. The erection of cofferdams could then be started at the following locations:—

I. The lock area on the Hopewell Cape shore.
II. The power-house and inlet gate area.
III. The outlet gate area on the east shore of the Memramcook. At this latter location certain preparatory excavation would be necessary in order to remove the overlying mud, and to prepare suitable foundations for the timber cofferdam.

The major portion of the outlet gate cofferdam structure will necessarily be the cellular type of steel sheet pile bulkheads, owing to the depth to ledge rock.

**Inlet and Outlet Gate Structures.**—Following the erection of necessary cofferdams, the construction of these main elements of the project must proceed expeditiously. Until the sills, piers, wing walls, and bulkhead sections of the gate structures have been completed it will not be possible to carry the rock-fill dams higher than the present low-water level in the two estuaries. In the outlet gate structure particularly, due to the depth of the gate sills, the gates must be erected and tested for operation before the protective cofferdams can be removed. It is herein considered that the placement of concrete in these structures could continue from April 1 to December 31 only during any construction year, and that such work must be suspended during the three colder winter months.

**Lock and Docks at Hopewell Cape.**—The lock and dock construction could be carried on independently of the work in progress at Fort Folly Point or on the gate structures. Excavation of the overlying mud could start immediately upon award of contract, with the necessary cofferdamming and concrete placing

following immediately from a separate and distinct plant set-up on the Hopewell Cape shore. Concreting during the months of January, February, and March would be suspended during any construction year.

**Rock-Fill Dam Construction.**—Upon completion of the gate structures, the placement of rock fill, core material, etc., in the two rock-fill dams could again be started and carried to completion following the sequence of operations described heretofore under Section No. 8.

**Power-house.**—The start of construction on the power-house proper would be governed by the speed with which cofferdams could be erected, and also by the speed of excavation operations in the head channel. The progress of the power-house construction will depend to a large extent on whether or not excessive leakage develops during the deep rock excavation in the draft tube areas. As mentioned under the subsections dealing with the gates and lock, concreting would be suspended during the colder winter months.

**Underwater Excavation.**—Underwater excavation or excavation in the "wet" will be necessary in certain areas, particularly under and adjacent to the cofferdam structures downstream from the power-house area, the inlet gate area, and adjacent to certain portions of the cofferdam structures both upstream and downstream from the outlet gate area.

**Installation of Hydraulic and Electrical Equipment.**—It will not be necessary to describe at length the various operations dealing with the installation of machinery. On the proposed construction schedule shown on Plate No. 53, the time element relative to the ordering, delivery and erection of such machinery is indicated graphically.

**Concrete Aggregate.**—The supply of suitable concrete aggregate is a very important factor affecting the whole project, and unfortunately will not be subject to definite analysis until a good deal of exploration for suitable gravel deposits has been carried out within the radius of economical truck haul. Deposits of suitable concrete aggregates are not plentiful in New Brunswick. It definitely is known that aggregates may be obtained in the vicinity of Sussex, and in the cost estimates, which appear hereafter, the cost of train-hauling aggregate to the site has been included.

**Proposed Construction Schedule.**—Plate No. 53 shows, graphically, a study of the proposed construction schedule covering the main work items associated with the project. The more important facts indicated by the schedule may be listed as follows:—

(A) The first generating unit may be brought into commercial operation at the end of the sixth year of the construction period. At this same time, however, fourteen units could be brought into commercial operation if desired.

(B) The key structures, in so far as the time element is concerned, are the inlet and outlet gate structures. The schedule indicates that the concrete work in these structures would not be completed until the summer of the fourth year of construction. This is primarily due to the extensive cofferdamming necessary. As set out in the estimates of cost which follow hereafter, ordinary rock excavation is estimated at $1.25 per cubic yard, and subaqueous rock excavation is estimated at $7 per cubic yard. This differential of $5.75 per cubic yard if applied to the 2,950,000 cubic yards of rock excavation in the outlet gate structure would amount to $16,962,500. The cost of cofferdams and pumping for this same structure is estimated to be $4,236,500, and the saving in net capital cost therefore by cofferdamming the outlet gate structure amounts to $12,726,000.

Until the gate structures are completed and in operation, it would not be possible to carry the rock-fill structures above the low-water level in the two estuaries. Therefore, the main construction program associated with the rock-fill dams cannot begin until September 1 of the fourth year of the construction

period. After this date, an over-all period of two years and four months has been allowed to complete the fills, drive the necessary lower piling cut-off and construct the upper concrete core. This period is none too long, considering the abnormal conditions under which this work must be performed. The construction of these rock-fill structures will be both difficult and hazardous. The ever-changing tides, the occasional storms, and the cold winter period of two seasons, during which this work must be carried on, all affect the over-all time element to a great extent.

(C) The lock and docks adjacent to Hopewell Cape may be completed and in operation at the end of the second year of construction. Ships may thereafter proceed through the lock without hazard, and without interference to the work in progress elsewhere on the project.

(D) The extremely large amount of dredging necessary in the Memramcook basin (225,000,000 cubic yards) may, of course, be carried on over a comparatively long period of time. The schedule indicates that dredging would be proceeded with during a 10-month period each year. The amount of dredging equipment necessary, and the over-all time schedule for this item, would depend on the system demand, and could be speeded up or slowed down to conform with the installation of generating equipment in the powerhouse.

As mentioned previously, the schedule indicates that the first generating unit could be brought into commercial operation at the end of the sixth year of the construction period, and also indicates that 14 units could be assembled, ready for commercial operation, by the same date. In the event of actual construction being proceeded with, the matter of turbine and generator contracts and installation would necessarily be subject to further special study, because the number of units provided for initial operation would depend on the available and prospective power contracts at that particular time, as would likewise the amount of dredging necessary in the Memramcook basin.

## 11. CAPITAL COST OF PRIMARY POWER

Estimated capital costs may be summarized as follows, with the detailed estimates comprising this summation of costs following thereafter:—

| Item | Main Division of Project | Estimated Cost |
|---|---|---|
| (a) | General Items Chargeable to Capital Accounts | $ 3,025,000 00 |
| (b) | Lock and Dock, Adjacent to Hopewell Cape | 3,102,090 00 |
| (c) | Rock-fill Dam—Petitcodiac River | 8,431,390 00 |
| (d) | Rock-fill Dam—Memramcook River | 4,414,300 00 |
| (e) | Inlet Gate Structure | 7,827,350 00 |
| (f) | Outlet Gate Structure | 11,875,230 00 |
| (g) | Head Channel | 17,978,750 00 |
| (h) | Power-house Substructure | 14,371,375 00 |
| (i) | Power-house Superstructure | 1,180,000 00 |
| (j) | Hydraulic Machinery | 9,570,000 00 |
| (k) | Electrical Machinery | 10,716,795 00 |
| (l) | Station and Miscellaneous Services | 310,500 00 |
| (m) | Dredging | 29,697,700 00 |
| Net Total | | $122,500,480 00 |
| Engineering and Contingencies—15% | | 18,375,072 00 |
| Sub-total | | $140,875,552 00 |
| Interest during construction based on an annual charge of 3½% for an average period of 3 years | | 14,791,933 00 |
| Grand Total | | $155,667,485 00 |

Cost per horse-power for 76,000 horse-power of continuously available power at 100% load-factor ........ $ 1,946 37

Item (*a*)—General Items Chargeable to Capital Accounts:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Construction Power | ........ | ........ | $ 120,000 00 |
| Construction Camps—Net Loss | ........ | ........ | 1,500,000 00 |
| Construction Railway | ........ | ........ | 500,000 00 |
| Construction Roads | ........ | ........ | 20,000 00 |
| Construction Yards | ........ | ........ | 20,000 00 |
| New Wharf at Coles Head | ........ | ........ | 50,000 00 |
| Sewage Disposal Plant and New Intercepting Sewers at Moncton | ........ | ........ | 600,000 00 |
| Damages—Dyked Lands—Petitcodiac River | ........ | ........ | 75,000 00 |
| Clearing and Removal of Lighthouse, etc., at Fort Folly Point | ........ | ........ | 40,000 00 |
| Model Tests | ........ | ........ | 100,000 00 |
| Net Total | | | $3,025,000 00 |

Item (*b*)—Lock and Dock, Adjacent to Hopewell Cape:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Timber Cofferdams—Lock Section | 24,000 cu. yd. | $ 10 50 | $ 252,000 00 |
| Pumping—Lock Section | ........ | ........ | 10,000 00 |
| Earth Excavation | 146,000 cu. yd. | 0 40 | 58,400 00 |
| Rock Excavation | 92,500 " | 2 00 | 185,000 00 |
| Concrete—Lock Floor | 3,400 " | 13 00 | 44,200 00 |
| Concrete—Lock Walls | 27,400 " | 17 25 | 472,650 00 |
| Concrete—Dock Walls | 51,300 " | 19 30 | 990,090 00 |
| Reinforcing Steel | 100 tons | 120 00 | 12,000 00 |
| Grouting—Lock Area | ........ | ........ | 25,000 00 |
| Gravel Fill—Dock Area | 177,000 cu. yd. | 0 75 | 132,750 00 |
| Lock Gates | 4 sets | ........ | 630,000 00 |
| Emergency Gates | 1 set | ........ | 100,000 00 |
| Highway Bridge | ........ | ........ | 150,000 00 |
| Haulage Units, Channel Markers, Lighting and Miscellaneous | ........ | ........ | 40,000 00 |
| Net Total | | | $3,102,090 00 |

Item (*c*)—Rock-fill Dam—Petitcodiac River:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Rock Fill—Quarry-run and Large—Placed by Marine Equipment | 2,030,000 cu. yd. | $ 1 75 | $3,552,500 00 |
| Rock Fill—Placed by Truck | 1,530,000 " | 1 25 | 1,912,500 00 |
| Fine Core Fill—Marine Equipment | 770,000 " | 1 00 | 770,000 00 |
| Fine Core Fill—Truck Placed | 330,000 " | 0 75 | 247,500 00 |
| Steel Sheet Piling—Bottom | 302,240 sq. ft. | 3 50 | 1,057,840 00 |
| Steel Sheet Piling—Top | 256,000 " | 1 75 | 448,000 00 |
| Trench Excavation | 23,600 cu. yd. | 4 00 | 94,400 00 |
| Concrete Core Wall | 31,300 " | 10 50 | 328,650 00 |
| Fencing, Lighting, Trimming, etc | ........ | ........ | 20,000 00 |
| Net Total | | | $8,431,390 00 |

Item (*d*)—Rock-fill Dam—Memramcook River:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Rock Fill—Quarry-run and Large—Placed by Marine Equipment | 880,000 cu. yd. | $ 1 75 | $1,540,000 00 |
| Rock Fill—Placed by Truck | 880,000 " | 1 25 | 1,100,000 00 |
| Fine Core Fill—Marine Equipment | 380,000 " | 1 00 | 380,000 00 |
| Fine Core Fill—Truck Placed | 160,000 " | 0 75 | 120,000 00 |
| Steel Sheet Piling—Bottom | 177,000 sq. ft. | 3 40 | 601,800 00 |
| Steel Sheet Piling—Top | 193,000 " | 1 75 | 337,750 00 |
| Trench Excavation | 18,000 cu. yd. | 4 00 | 72,000 00 |
| Concrete Core Wall | 23,500 " | 10 50 | 246,750 00 |
| Fencing, Lighting, Trimming, etc | | | 16,000 00 |
| Net Total | | | $4,414,300 00 |

Item (*e*)—Inlet Gate Structure:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Timber Cofferdams | 243,750 cu. yd. | $10 50 | $2,559,375 00 |
| Pumping | | | 50,000 00 |
| Rock Excavation—Dry | 364,500 cu. yd. | 1 25 | 455,625 00 |
| Rock Excavation—Subaqueous | 46,000 " | 7 00 | 322,000 00 |
| Concrete—Piers | 73,500 " | 17 50 | 1,286,250 00 |
| Concrete—Floor and Apron | 12,020 " | 13 00 | 156,260 00 |
| Concrete—Deck and Bridge | 5,070 " | 20 00 | 101,400 00 |
| Concrete—Wing Walls | 20,800 " | 17 25 | 358,800 00 |
| Reinforcing Steel | 1,744,000 lbs. | 0 06 | 104,640 00 |
| Gates and Hoists | | | 2,288,000 00 |
| Emergency Gates | 2 sets | | 85,000 00 |
| Gantry Crane | | | 50,000 00 |
| Railings, Lighting and Miscellaneous | | | 10,000 00 |
| Net Total | | | $7,827,350 00 |

Item (*f*)—Outlet Gate Structure:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Cofferdams—Cellular Steel Piling and Timber | 6,900 l. ft. | $585 00 | $ 4,036,500 00 |
| Pumping | | | 200,000 00 |
| Rock Excavation—Dry | 2,950,000 cu. yd. | 1 25 | 3,687,500 00 |
| Concrete—Piers | 101,000 " | 17 50 | 1,767,500 00 |
| Concrete—Floor and Apron | 13,360 " | 13 00 | 173,680 00 |
| Concrete—Deck only | 1,580 " | 20 00 | 31,600 00 |
| Concrete—Wing Walls | 31,400 " | 17 25 | 541,650 00 |
| Reinforcing Steel | 1,830,000 lbs. | 0 06 | 109,800 00 |
| Gates and Hoists | | | 1,232,000 00 |
| Emergency Gates | 2 sets | | 45,000 00 |
| Gantry Crane | | | 40,000 00 |
| Railings, Lighting and Miscellaneous | | | 10,000 00 |
| Net Total | | | $11,875,230 00 |

Item (*g*)—Head Channel:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Cofferdams | | | Included elsewhere |
| Rock Excavation Placed Direct in Rock-fill Dams | | Included under Rockfill Dams | Included elsewhere |
| Rock Excavation—Stockpiled or Otherwise Disposed of | 14,383,000 cu. yd. | $ 1 25 | $17,978,750 00 |
| NET TOTAL | | | $17,978,750 00 |

Item (*h*)—Power-house Substructure:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Cofferdams | 386,750 cu. yd. | $10 50 | $ 4,060,875 00 |
| Pumping | | | 100,000 00 |
| Rock Excavation—Above Elev.-20 | Included under Head Channel | | |
| Rock Excavation—Below Elev.-20 (Dry)—(Substructure, Draft Tubes and Tailrace) | 934,000 cu. yd. | 2 25 | 2,101,500 00 |
| Rock Excavation—Subaqueous | 52,000 " | 7 00 | 364,000 00 |
| Concrete | 340,000 " | 18 00 | 6,120,000 00 |
| Reinforcing Steel | 20,000,000 lbs. | 0 06 | 1,200,000 00 |
| Miscellaneous Structural Steel | | | 60,000 00 |
| Air, Oil, Water and Drain Piping | | | 300,000 00 |
| Headworks and Draft Tube Emergency Gates | 2 sets each | | 20,000 00 |
| Headworks Gantry | | | 25,000 00 |
| Tailrace Gantry | | | 20,000 00 |
| NET TOTAL | | | $14,371,375 00 |

Item (*i*)—Power-house Superstructure:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Structural Steel | 4,000,000 lbs. | $ 0 10 | $ 400,000 00 |
| Concrete | 12,000 cu. yd. | 40 00 | 480,000 00 |
| Reinforcing Steel | 700,000 lbs. | 0 06 | 42,000 00 |
| Roof and Roofing | 140,000 sq. ft. | 0 70 | 98,000 00 |
| Windows, Doors, Partitions, etc | | | 45,000 00 |
| Steel Stairs and Railings | | | 15,000 00 |
| Floor Finish | 100,000 sq. ft. | 0 60 | 60,000 00 |
| Paint and Finish | | | 30,000 00 |
| Miscellaneous | | | 10,000 00 |
| NET TOTAL | | | $ 1,180,000 00 |

Item (*j*)—Hydraulic Machinery:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Turbines and Governors, Erected | 30 | $319,000 00 | $ 9,570,000 00 |
| NET TOTAL | | | $ 9,570,000 00 |

Item (*k*)—Electrical Machinery:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Generators | 30 | $282,000 00 | $ 8,460,000 00 |
| Generators—Spare Parts | | | 36,000 00 |
| Low-tension Switching | | | 515,180 00 |
| Cables, Conduits and Wiring | | | 240,975 00 |
| Bus Structures | | | 14,400 00 |
| Auxiliary Service Equipment | | | 81,780 00 |
| Grounding System | | | 12,240 00 |
| Transformers | | | 675,000 00 |
| High-tension Switching, including Switching Structure | | | 681,220 00 |
| NET TOTAL | | | $10,716,795 00 |

Item (*l*)—Station and Miscellaneous Services:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Cranes | 2 | $ 45,000 00 | $ 90,000 00 |
| Power-house Lighting | | | 65,100 00 |
| Storage Battery and M.G. Sets | | | 3,400 00 |
| Signalling System | | | 45,000 00 |
| Pumps and Motors | | | 25,000 00 |
| Air Compressor and Auxiliaries | | | 12,000 00 |
| Service Water Equipment | | | 10,000 00 |
| Oil Storage and Filters | | | 25,000 00 |
| Sanitary and Plumbing | | | 10,000 00 |
| Heating and Ventilating | | | 25,000 00 |
| NET TOTAL | | | $ 310,500 00 |

Item (*m*)—Dredging:—

| — | Quantity | Unit Price | Amount |
|---|---|---|---|
| Memramcook Basin | 225,000,000 cu. yd. | $ 0 13 | $29,250,000 00 |
| Outlet Gate Structure | 4,070,000 " | 0 11 | 447,700 00 |
| NET TOTAL | | | $29,697,700 00 |

Estimates of capital cost as shown are based on a complete and total installation of 30 units, and are based on complete and final dredging of the Memramcook basin. The construction schedule indicates that all permanent works would be complete by the end of the sixth year, but also provides, in the alternative, that the dredging contract, together with the contracts for the progressive installation of hydraulic and electrical machinery, may be extended indefinitely, depending entirely on market conditions for power at the end of the 6-year construction period, and subsequent thereto. For reasons outlined in Section No. 7 above, "Design and Characteristics of Electrical Installation", the installation of hydraulic and electrical machinery must, for estimating purposes, progress in increments of 5 complete units, or a total of 45,000 kva.

Therefore, the cost of permanent works and generating equipment may be segregated as follows into two main divisions:—

FIRST: Cost of permanent works and equipment necessary for the installation of the first 5 units (45,000 kva.) is as follows:—

| | |
|---|---|
| (*a*) General Items Chargeable to Capital Account........ | $3,025,000 00 |
| (*b*) Lock and Dock, adjacent to Hopewell Cape.......... | 3,102,090 00 |
| (*c*) Rock-fill Dam—Petitcodiac River.................... | 8,431,390 00 |
| (*d*) Rock-fill Dam—Memramcook River ................ | 4,414,300 00 |
| (*e*) Inlet Gate Structure ................................ | 7,827,350 00 |
| (*f*) Outlet Gate Structure................................ | 11,875,230 00 |
| (*g*) Head Channel ......................................... | 17,978,750 00 |
| (*h*) Power-house Substructure ........................... | 14,371,375 00 |
| (*i*) Power-house Superstructure—for 5 units, control and erection bays ..................................... | 248,000 00 |
| (*j*) Hydraulic Machinery—5 units........................ | 1,595,000 00 |
| (*k*) Electrical Machinery—5 units........................ | 2,034,595 00 |
| (*l*) Station and Miscellaneous Services.................. | 191,250 00 |
| (*m*) Dredging, necessary for first 5 units................ | 4,447,700 00 |
| Net Total ................................................ | $79,542,030 00 |
| Engineering and Contingencies—15 per cent............. | 11,931,304 00 |
| Sub-total ................................................ | $91,473,334 00 |
| Interest during construction based on an annual charge of 3½ per cent for an average period of 3 years.......... | 9,604,700 00 |
| Grand Total ................................................ | $101,078,034 00 |

SECOND: Cost of permanent works and equipment necessary for the installation of each subsequent block of 5 units (45,000 kva.) is as follows:—

| | |
|---|---|
| (*i*) Power-house Superstructure—5 units.................. | $ 185,000 00 |
| (*j*) Hydraulic Machinery—5 units........................ | 1,595,000 00 |
| (*k*) Electrical Machinery—5 units........................ | 1,772,440 00 |
| (*l*) Station and Miscellaneous Services.................. | 23,850 00 |
| (*m*) Dredging—pro-rated for 5 units...................... | 5,000,000 00 |
| Net Total ................................................ | $8,576,290 00 |
| Engineering and Contingencies—15 per cent............. | 1,286,443 00 |
| Sub-total ................................................ | $9,862,733 00 |
| Interest during construction, say—1 year at 3½ per cent..... | 345,195 00 |
| Grand Total ................................................ | $10,207,928 00 |

**Fixed and Variable Estimated Costs.**—The various items comprising the foregoing detailed estimates of costs are all susceptible of direct and rational computation and analysis, with the exception of the volume of rock sinkage, and fine core material sinkage, under the normal base elevation of the Petitcodiac and Memramcook rock-fill dams.

The estimates include what is considered to be reasonable provision for a total combined rock sinkage in both dams of 2,200,000 cubic yards, and a total combined fine core material sinkage in both dams of 492,000 cubic yards. Applying these latter figures to the average estimated costs of $1.50 per cubic yard for the rock and $0.875 for the fine core material, the following totals are derived:—

| | |
|---|---|
| 2,200,000 cu. yd. of rock sinkage at $1.50................. = | $3,300,000 00 |
| 492,000 cu. yd. of fine core material sinkage at $0.875...... = | 430,500 00 |
| Net Total ................................................ | $3,730,500 00 |
| Engineering and Contingencies—15 per cent.................. | 559,575 00 |
| Sub-total ................................................ | $4,290,075 00 |
| Interest during construction.................................. | 175,000 00 |
| Grand Total ................................................ | $4,465,075 00 |

The grand total estimated cost for the complete development as shown heretofore amounts to \$155,667,485. Therefore the percentage of fixed costs as applied to this total would amount to 97·13 per cent, and the percentage of variable cost would amount to 2·87 per cent. In other words, if there were no sinkage at all under the base of the two dams, the total estimated cost of the project could be set out to be \$151,202,410. The items comprising this latter total as mentioned above are based upon rational mathematical computation and analysis.

## 12. ANNUAL COST OF PRIMARY POWER

**Precedent.**—Inasmuch as there is no precedent for the establishment and operation of a project of the type and magnitude of that covered by this report, it follows that the annual cost of such operation must include considerations which differentiate it, in some essential particulars, from the normal type of publicly financed hydro-electric development. The various elements of annual cost are however identical, and include Operation; Maintenance and Repairs; Depreciation; Interest; Sinking-fund, and Contingency Reserve. In the present instance, all of these items have a special significance of their own, with the single exception of Interest.

**Interest.**—It may be assumed that any publicly financed project of this general type can obtain capital funds at a rate not exceeding 3½ per cent per annum.

**Sinking-fund.**—The sinking-fund obligation for projects of this kind is usually based upon 20 to 30-year maturities, but in view of the fact that (*a*) load development in this case will probably be less rapid than the average, and (*b*) that the support of public credit will simplify refunding operations, it is considered reasonable in this instance to fix the sinking-fund obligation on the basis of a 40-year term, compounding at 2½ per cent per annum, or the equivalent of 1·48 per cent per annum against total capital cost.

**Depreciation.**—The function of a depreciation reserve is to maintain the project, as a whole, in a state of "newness" while the sinking-fund is liquidating the bonded debt. In an installation of the type now under consideration a blanket charge of 1·5 per cent on total capital cost is usually sufficient to accomplish the intended purpose. In this instance an abnormally large turbine, generator, and regulating gate installation, with accessories, would tend to increase this charge, but on the other hand there is also an abnormally large investment in non-depreciable and low depreciation items, such as earth and rock excavation, rock-fill dams and concrete structures, which would tend to restore the balance, and thereby justify the use of a 1·5 per cent depreciation rate for the present purpose.

**Contingency Reserve.**—The primary purpose of this reserve is to assist in stablizing the rate structure, and secondarily as protection against the accident hazard. A normal allowance under this head is 1 per cent. In this case the accident hazard is normal, but the circumstances appear to be such as to require the accumulation of a substantial rate-stabilization reserve, so that for the present purpose the charge under this head will be fixed at 1·25 per cent.

**Maintenance and Repairs.**—This is a current charge which can normally be covered by a blanket charge of 1 per cent on capital cost. In this case, the large mechanical and electrical installation per horse-power of production, together with the difficulty in predicting the effect upon it of sea water, justifies the assumption of a charge of 1·25 per cent.

**Operation.**—There is no statistical precedent whatever by which to measure the probable cost of operating this project for the production of power of the amounts and under the conditions described in the report. Ordinarily this charge ranges upward from a minimum of 0·75 per cent.

In this case, however, the daily regimen of operation must, of necessity, be a new creation, and provide for conditions hitherto non-existent in hydro-electric plant operation. And beyond this is the obvious necessity of employing a greater-than-normal number of men per shift for the purely routine handling of the proportionately large installation of generating units, gates and auxiliary and control equipment. The charge under this head cannot safely be assumed at less than 1·25 per cent.

**Total Annual Charge.**—Summing up the above, the total annual charge against capital cost becomes:—

| | Per cent |
|---|---|
| Interest | 3·50 |
| Sinking-fund | 1·48 |
| Depreciation | 1·50 |
| Contingency Reserve | 1·25 |
| Maintenance and Repairs | 1·25 |
| Operation | 1·25 |
| Total Annual Charge | 10·23 |

**Total Annual Cost.**—The total annual cost of the completed development (30 units) amounts to $15,924,784 on the basis of the annual charges as set out above.

## 13. SUPPLEMENTARY AND ALTERNATIVE STEAM POWER

The installation described in the foregoing pages of this report has a peak capacity, in so far as equipment is concerned, of 280,500 turbine-horsepower. The equivalent of this quantity stated in electrical power at the generator terminals is 270,000 electrical-horsepower, or 201,000 kilowatts. As already pointed out, however, the period of availability of this peak output capacity is wholly dependent upon the periods of tidal variation, and is therefore ungovernable. The probability of coincidence with the demand peak of any normal power system is only once in several thousand tidal cycles.

To utilize this varying power potential to the fullest degree, therefore, a supplementary fully-controllable prime mover installation must be provided.

Doubtless, in the conception of the Petitcodiac tidal scheme, a community or district was visualized which would provide a market for the potential power and energy of the completed development. For the purposes of this report, this particular district has been assumed as one which would have a winter peak demand equal to the maximum peak capacity of the development. The type of demand assumed is one which would be normal to a metropolitan district with the usual percentage of ordinary industrial load, and a considerable demand from heavy industry. Curve "A" on Plate No. 54 is typical of the load demand for an average week for a district or community such as described. This curve represents the demand of a typical week either in spring or in autumn. The unit of demand is expressed in turbine-horsepower so that the curves on Plate No. 54 will be readily comparable to the other Plates in this report. The winter peak demand would be somewhat higher than shown, and the summer peak somewhat lower. To illustrate the difference between the winter peak and the summer peak, the curves on Plate No. 55 have been prepared.

For the purpose of making an economic study and determining the annual operating costs, the curves on Plate No. 54 have been used, since they represent average conditions.

Inspection will show quite clearly that for the load demand assumed, considerable energy would have to be developed by some other source than the hydro-electric plant. The amount of this supplementary energy would vary considerably from day to day and from season to season, in fact, the irregularity would be such that any definite operating schedule for the units of the hydro-electric plant would be quite difficult to project.

The instantaneous power deficiencies for the hypothetical week represented by the curves on Plate No. 54 can be readily determined by measuring the distance between curves "A" and "B". The energy deficiencies are represented by the black areas between these curves. Dotted areas indicate potential hydro-electric energy which could not be made use of.

In order to "firm up" the power available from the hydro-electric plant, so as to meet a winter peak demand of 280,500 turbine-horsepower, it would be necessary to provide a modern steam power plant, having an installed capacity of 150,000 kilowatts. Such a plant, built to modern standards but not including spare units such as would be required in a base-load plant, would cost approximately $22,000,000.

Based on the preceding discussion it is evident that first cost and annual operating estimates must be prepared and presented herein in a form which will permit a comparative analysis of the merits of at least three schemes of installation and operation. It would seem that the logical alternatives would be as follows:—

(1) Installation of a hydro-electric plant as described in this report and operation in a manner that would allow the plant to supply the maximum possible commercial power and energy demand continuously. The output rating of the plant under these conditions would be 73,000 electrical-horsepower, or 54,500 kilowatts.

(2) Installation of a hydro-electric plant supplemented by a 150,000-kilowatt steam plant and operation in such a manner that the full power capacity of the hydro-electric plant would be utilized whenever tidal conditions made it available and so that the steam plant would supply all deficiencies in power and energy which would result from failure of the demand peaks and hydro-electric plant capacity peaks to coincide. The output rating of the combined plants under these conditions would be about 270,000 electrical-horsepower, or 201,000 kilowatts.

(3) Installation of a modern steam electrical generating station having a commercial peak capacity of 270,000 electrical-horsepower, or 201,000 kilowatts, which capacity would be equivalent to the intermittent peak capacity of the hydro-electric plant alone, or the commercial peak capacity of the combined hydro-electric and peak load steam plants mentioned under Item 2.

For the purposes of this comparison it is assumed that the maximum peak demand of the station would be required and the estimated costs are based upon this premise. The total capital expenditures required therefore for each of the above schemes would be as follows:—

Estimates of Capital Expenditures

| | | |
|---|---|---|
| 1. Hydro-electric Plant | | $155,667,485 00 |
| 2. Combined Hydro-electric and Peak Load Steam Plant— | | |
| Hydro-electric Plant | $155,667,485 00 | |
| Steam Plant | 22,000,000 00 | $177,667,485 00 |
| 3. Steam Plant, 200,000 Kilowatts, plus spares | | $ 35,000,000 00 |

## Estimates on Annual Operating Costs

(1) The annual operating costs of the hydro-electric plant have already been estimated at $15,924,784, which amount on the basis of 73,000 electrical-horsepower represents a total cost amounting to $218.15 per electrical-horsepower per year of available commercial capacity. The energy output would be 291,000,000 kilowatt-hours at a unit cost of 54·7 mills.

(2) The annual cost of operating the hydro-electric plant in combination with the peak load steam-electric plant is as detailed in the estimate following. It is based on full-load operation in accordance with the load curve on Plate No. 54 and the total quantity of fuel is based upon that amount required to produce the full amount of make-up steam electric energy, plus an allowance for banking and heating during the periods when the hydro-electric plant would be capable of meeting the load demand by itself:—

| | | |
|---|---|---|
| Annual Operation Hydro Plant | | $15,924,784 00 |
| Annual Operation Steam Plant— | | |
| Fixed Charges at 10 per cent | $2,200,000 00 | |
| Operating Labour | 235,000 00 | |
| Maintenance—Labour and Material | 75,000 00 | |
| Supplies | 20,000 00 | |
| Fuel—47,000 Tons at $7 | 329,000 00 | 2,859,000 00 |
| Total | | $18,783,784 00 |

Thus the total cost of carrying the assumed peak load of 270,000 electrical-horsepower for a year would be $18,783,784 which amounts to $69.57 per horsepower per year. The equivalent energy cost for 1,076,300,000 kilowatt-hours per annum would be 17·45 mills. This amount of energy has been calculated from curve "A" on Plate No. 54, which curve represents a demand having a weekly load-factor of 68·8 per cent. The annual load-factor would be 61·0 per cent.

(3) The annual cost of operating a modern steam plant to provide the same services outlined for the combined hydro-electric and peak-load steam plant is shown in detail below:—

| | | |
|---|---|---|
| Annual Operation Steam Plant— | | |
| Fixed Charges at 10 per cent | $3,500,000 00 | |
| Operating Labour | 275,000 00 | |
| Maintenance—Labour and Material | 185,000 00 | |
| Supplies | 50,000 00 | |
| Fuel—490,000 Tons at $7 | 3,430,000 00 | |
| Total | | $7,440,000 00 |

This amount of $7,440,000 represents the total cost of supplying the load demand at a maximum rate of 270,000 electrical-horsepower and therefore the cost per electrical-horsepower per year would be about $27.55. The energy costs for the same number of kilowatt-hours as in Item 2 would be 6·91 mills.

For clarification and readier reference, the significant figures relating to the three suggested schemes of installation and operation have been summarized in Table A (page 47).

In each of the proposed schemes a typical load demand has been assumed which would have a peak corresponding to the maximum peak output capacity of the generating facilities. Schemes 2 and 3 are identical but on account of the limitations of the controllable output capacity of the proposed hydro-electric installation of Scheme 1, a reduced load-demand curve was used. This curve is identical in characteristics with the load curve for the other two schemes except that its ordinates have been reduced in magnitude so that the maximum winter peak would not exceed 73,000 electrical-horsepower. This is the quantity which, as stated before, is the limit of the dependable and commercial peak output of the proposed hydro-electric station when operating as an isolated plant.

The choice of full ultimate installation and maximum ultimate demand as a basis for comparative estimates was made in order that results most favourable to the proposed hydro-electric installation would be shown. The figures in the three columns of Table A indicate these results and there should be no need for further explanation. Any estimates based upon partial installations of generating capacity would show differences in capital and operating costs per unit of output greater than in the table. This is obvious since the greater proportion of the proposed hydro-electric plant must be completed regardless of the number of generating units installed while the steam plants could be built in increments more or less in proportion to the load demand.

TABLE A

Summary of Costs and Pertinent Data for Comparison of Three Installation Schemes

| — | No. 1 Hydro-Electric Only | No. 2 Hydro-Electric and Peak Load Steam | No. 3 Steam Only |
|---|---|---|---|
| Peak demand—e.h.p. | 73,000 | 270,000 | 270,000 |
| Energy—millions k.w.h. per annum | 291·0 | 1076·3 | 1076·3 |
| Weekly load-factor—per cent | 68·8 | 68·8 | 68·8 |
| Annual load-factor—per cent | 61·0 | 61·0 | 61·0 |
| Capital investment | $155,667,485 00 | $177,667,485 00 | $35,000,000 00 |
| Annual cost—Total | 15,924,784 00 | 18,783,784 00 | 7,440,000 00 |
| Cost per e.h.p. per year | 218 15 | 69 57 | 27 55 |
| Cost per k.w.h.—mills | 54·7 | 17·45 | 6·91 |

## 14. SUMMARY

The essential points of the foregoing discussions and explanations are summarized herewith.

(1) The tidal range at the confluence of the Petitcodiac and Memramcook Rivers is among the very highest in the world. Height of tidal range constitutes one of the two fundamental conditions necessary for the development of power, and accordingly the proposed site surpasses most others in this respect.

(2) The second fundamental condition demands that there be extensive basins capable of storing water during the tidal cycle. This requirement is fulfilled at the proposed site by the tidal estuaries of the Petitcodiac and Memramcook Rivers, either or both of which can be used for storage purposes.

(3) The co-existence, in extremely favourable form, of the above two prerequisite conditions makes the Petitcodiac site outstanding from the viewpoint of its power potentialities.

(4) The fact that two estuaries are available for use as tidal basins, coupled with the further fact that the site allows them to be used as high and low-level basins, makes possible the generation of continuous power, during the entire 24 hours of each day. Continuous power, as opposed to the intermittent power produced by a single basin arrangement, is the only kind possible for commercial use where the tidal power plant forms any large proportion of the total connected generating capacity.

(5) Structures can be designed to develop the full power potential of the site; the chief features of such structures are shown on the Plates of this report and are described in the text.

(6) It is feasible to construct the required works, although some portions of them will involve unusual difficulties. The solutions proposed are described in Section No. 8 of the report.

(7) The daily tidal cycle is approximately 24 hours and 50 minutes whereas normal power loads have a cyclical variation of 24 hours. No effective correlation matching power production with power demand can therefore be made. Practical alternatives are described at length in the body of the text.

(8) The maximum continuous power potential, at 100 per cent load-factor, will vary with the height of the tides, ranging from a minimum of 76,000 turbine-horsepower to a maximum of 280,500 horse-power. Although this extreme variation will take place over a period of about 18 years, changes in tidal height and hence in available continuous power potential will vary between intermediate highs and lows every lunar month.

(9) Similarly, with the proposed station operating to produce the maximum amount of energy instead of the maximum amount of continuous power, generation would vary from an extreme minimum of 1,500,000 kilowatt-hours to an extreme maximum of 4,850,000 kilowatt-hours per 24 hours, depending upon the tidal range. During the 18-year cycle the average production would be 3,600,000 kilowatt-hours per day.

(10) The estimated time required to construct all the permanent works is 6 years. Of the proposed total of 30 generating units, any number up to 14 can be installed during this period. More units can be added as and when required.

(11) The total estimated capital cost of the complete development, including interest during the construction period at 3½ per cent per annum, is $155,667,485. A considerable portion of this cost is due to the extremely unfavourable foundation conditions at the site.

(12) Of the above, $101,078,034, or 65 per cent of the estimated total is the estimated cost of all the permanent works, together with 5 generating units and the necessary power-house superstructure.

(13) The estimated annual costs for the complete tidal power plant are estimated to be $15,924,784. The commercial firm power is 73,000 electrical-horsepower and on the basis of these figures the production cost is $218.15 per electrical-horsepower per year.

The potential average annual energy production of tidal power is approximately 1,310,000,000 kilowatt-hours. If all of this could be sold, the production cost would be 12·2 mills per unit. However, on the basis of supplying 73,000 electrical-horsepower of firm commercial power with an annual load-factor of 61·0 per cent, as described in Section No. 13, the annual production amounts to 291,000,000 kilowatt-hours at a cost of 54·7 mills per unit.

(14) Supplementary steam-generated power is nceessary to firm up the deficiency in tidal-generated power during periods of medium and low tides. The cost of a steam-electric plant, having sufficient capacity to supply all deficiencies up to the maximum necessary for the full output of 280,500 turbine-horsepower, or 270,000 electrical-horsepower, has been estimated as $22,000,000. The combined capital cost per installed electrical-horsepower would be $658 and the estimated annual cost $69.64 per electrical-horsepower per year. With an annual load-factor of 61·0 per cent, the annual production in this case would be 1,076,300,000 kilowatt-hours at a unit cost of 17·45 mills.

(15) If steam-electric power alone were used to supply a normal metropolitan load having a peak demand corresponding to 270,000 electrical-horsepower at the generator terminals, the capital cost of the necessary generating station has been estimated as $35,000,000 and the annual costs as $7,440,000. The corresponding production costs are $27.55 per electrical-horsepower per year and 6·91 mills per kilowatt-hour. These figures compare directly with $69.64 and 17·45 mills, respectively, given in Item 14 above.

(16) The capital cost and power potential of the complete tidal power plant have been used in this report in order to show both the amount of power available and the unit costs, associated therewith, in their most favourable aspects. As will be seen from the figures of Item 12 above, the installation of only part of the ultimate generating capacity would result in much higher figures for the capital cost per horse-power and for the annual costs for each kilowatt delivered by the station. In other words, if the economics of the complete development are unfavourable, those of partial installation would be even more so.

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Respectfully submitted,

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Oct. 31, 1945.

## APPENDIX No. 1

### MOUNT ALLISON UNIVERSITY, SACKVILLE, NEW BRUNSWICK

### Report on Samples of the Petitcodiac River

| Sample | Density Without Silt Grams/CC | Density With Silt Grams/CC | Silt Per Litre | Parts of Silt Per Thousand Parts of Pure Water by Weight |
|---|---|---|---|---|
| 0′ | ............ | 1·0223 | ·1316 g. | ·1318 |
| 5′ | 1·0221 | 1·0222 | ·1272 g. | ·1274 |
| 10′ | ............ | 1·0223 | ·1834 g.<br>·1828 g. (check) | ·1837 |
| 15′ | 1·0222 | 1·0228 | ·2202 g. | ·2205 |
| 20′ | ............ | 1·0223 | ·1350 g. | ·1352 |
| 25′ | ............ | 1·0227 | ·2133 g. | ·2136 |

Temperature—18·5 degrees Centigrade in all cases.
In the density measurements the last place of decimals is not significant, i.e., cannot be guaranteed.
Sample marked 20′ obviously contained less silt by inspection.

## APPENDIX No. 2

### Weight of Petitcodiac River Water

| Depth Below Surface<br><br>Feet | River Water With Silt Removed<br><br>Weight in Lbs. Per Cubic Foot | River Water<br><br>Weight in Lbs. Per Cubic Foot |
|---|---|---|
| 0 | ............ | 63·720 |
| 5 | 63·707 | 63·714 |
| 10 | ............ | 63·720 |
| 15 | 63·714 | 63·751 |
| 20 | ............ | 63·720 |
| 25 | ............ | 63·745 |